INVESTIGATION ON THE USE OF EQUIVALENCY FACTORS FOR THE DESIGN AND EVALUATION OF FLEXIBLE AIRFIELD PAVEMENTS

THESIS

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Abstract

With over 1.5 billion square feet of airfield pavements in its portfolio, the U.S. Air Force has a vested financial interest in refining its design, maintenance, and inspection criteria to increase the efficiency of its infrastructure investment. As part of its strategic pavement assessment, the Air Force adopted a new design method (CBR-Beta) developed by the U.S. Army Corps of Engineers (USACE) since it more accurately represents the performance of flexible airfield pavements, particularly with newer, heavier aircraft. Supporting this adoption of the new method, this research primarily focused on evaluating the current set of equivalency factors in use by the Air Force and the USACE using a meta-analysis approach. Building on this initial success, the research shifted to analyzing the life-cycle costs of the various flexible pavement design methods relative to a common design standard to eliminate the problems associated with comparing the methods each with its own assumptions and processes. To further refine the predictability of the CBR-Beta method, the research analyzed the formulation of Frohlich's concentration factor. Additionally, the research assessed the possibility of expanding the empirical airfield data set with highway testing data. Ultimately, this research led to recommending new equivalency factors for stabilized layers, a new two-layer concentration factors model, an extension to CBR-Beta for highway pavements, and provides evidence to reformulate β as a stress-derived variable as opposed to failure-derived.

To the City Upon a Hill

To the Men and Women who Took to Arms to Protect Her Gates

To Those who Stood Fast and Suffered Long, Undaunted by the Odds or Fear of Danger

To Those Still Suffering

To Those that Gave Their Last Full Measure of Devotion

To Tyler and Dave

To All the Families Left Broken and in Need of Healing

To the Debt Owed by So Many to So Few

Here's a Toast

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Thomas M. Synovec

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INVESTIGATION ON THE USE OF EQUIVALENCY FACTORS FOR THE DESIGN AND EVALUATION OF FLEXIBLE AIRFIELD PAVEMENTS

I. Introduction

The purpose of this chapter is to introduce the research focus, summarize the background information, define the problem, establish research objectives, and present an overview of the research methodology and subsequent thesis chapters. In the background section, the chapter briefly introduces stabilized soils and equivalency factors and highlights previous research efforts in these fields. The brief introduction to the subject and previous research efforts serve as a baseline to the introduction of the overall research objective and accompanying secondary objectives. The chapter discusses the research approach utilized to address the primary and secondary objectives, including a discussion on research scope and limitations. The chapter concludes by aligning the research objectives with the three scholarly articles and a white paper contained in Chapters III through VI.

Background

This research focused on the use of equivalency factors within the Air Force's current flexible pavement design methodology to account for the inclusion of stabilized soils in airfield pavements. Equivalency factors allow engineers, particularly in contingency environments, to account for the improved performance of stabilized soils by substituting the stabilized soil for the more conventional, non-stabilized soil using the published factor for a respective soil (U.S. Army Corps of Engineers, 2001). Equivalency factors are proportionality constants that specify the ratio of substitution for a respective stabilized soil over a conventional soil.

The military currently has two design methods for flexible pavements as detailed in UFC 3-260-02, *Pavement Design for Airfields*, and UFC 3-250-01FA, *Pavement Design for Roads, Streets, Walks, and Open Storage Areas*. Both publications incorporate the use of equivalency factors to account for the design and construction of various types of base and subbase materials; these factors also apply to stabilized materials. Equivalency factors are also used in UFC 3-260-03, *Airfield Pavement Evaluation*, to perform pavement evaluations on airfields. However, most equivalency factors were developed based upon the Air Force's operational environment in the 1970s, as well as limited testing, and do not align with factors in use by other federal and state agencies. Therefore, subject matter experts familiar with their development and utilization have raised concerns about the accuracy and application of the equivalency factors (Personal Communication, 3 Jan 2013).

In 2010, the Air Force Civil Engineer Center (AFCEC) contracted a team of experts to evaluate its pavement program in its entirety; the Air Force and the Army utilize the same pavement design and evaluation standards with minor differences. The experts made several recommendations to the AFCEC to include a reevaluation of equivalency factors as they believed the current factors were overly conservative and did not accurately characterize the superior performance of stabilized soils (Monismith, Thompson, Leahy, & Zapata, 2010). In their concluding remarks, the team listed equivalency among its top recommendations to the AFCEC for corrective action.

Stabilized Soils

Subbase course materials are typically found in the local area; however, in the event that local materials cannot meet the gradation and strength requirements, materials for the subbase would have to be hauled to the site from a material supplier. On the other hand, base course materials are engineered and quarried to meet gradation and strength requirements; these soil materials do not typically occur naturally. Engineering these materials can be expensive, and in some cases require soil to be transported to the site from a distant supplier, further increasing the cost, and potentially the timeline, of the project. For contingency environments, accessibility and time may not allow the transportation of these better materials; therefore, in the 1950s, the U.S. Army Corps of Engineers (USACE) began investigating the use of stabilization techniques to improve the quality of local (respective to the construction site) soil (Ahlvin, 1991). Prior to this point, other transportation-related organizations were investigating soil stabilization; however, each of these organizations solely focused on applying this technique for highways.

Early attempts at soil stabilization focused on simply mixing higher quality soils with the lower quality, local soils; further testing revealed that chemical stabilization offered improved performance. Though the primary benefits for soil stabilization are facilitating the use of more economical local materials and reducing pavement thickness requirements, several additional benefits exist to include mitigating the effects of expansive soils and improving soil workability (U.S. Army Corps of Engineers, 2001). There are currently several different methods for soil stabilization; identifying the best method depends on the type of soil being stabilized. Table 1 highlights several of these stabilization techniques as well as the ideal soil for each respective technique; this table was created from information provided in UFC 3-260-02.

Table 1. Common Airfield Soil Stabilization Techniques (Adapted from U.S. Army Corps of Engineers, 2001)

	Stabilizing Agent	Most Suitable Soils
Most Common	Lime	Clayey soils with a plasticity index (PI) of 12 or more
	Portland Cement	Well-graded sandy gravels or gravelly sands with a spectrum of particle sizes
	Bituminous (Asphalt)	Granular materials with a PI less than 6 and with less than 12 percent fines (ideally); not to exceed 10 and 30 respectively
	Pozzolan and Slag	Granular materials, particularly effective with poorly graded materials

It is important to note that although chemically stabilized soils are the predominant alternative when considering soil stabilization, other methods exist that can achieve similar effects. These methods include mechanical and granular stabilization. Mechanical stabilization typically involves using geosynthetics on the subgrade to provide “bridging” over fine-grained soils (Federal Aviation Administration, 2009). The military has conducted research on geosynthetics and published guidance on its usage; however, using geosynthetics does not involve equivalency factors, thus it is not discussed. Granular stabilization involves the use of higher quality granular materials than a traditional base or subbase course, such as crushed limestone. The U.S. Department of Defense (DoD) accounts for higher quality granular base courses by reducing the minimum thickness criteria and reducing the subbase, as necessary, to account for the higher quality materials; this process is detailed in Chapter II. For higher quality granular subbase materials, the DoD uses an equivalency factor of 2.0 when the materials meet the requirements for a base course material (U.S. Army Corps of Engineers, 2001). In the event the higher quality subbase material does not meet the gradation and strength requirements for a base course, no equivalency factors are used; however, the wearing and base course thicknesses are ultimately reduced to coincide with the reduced required thickness above the subbase due to the subbase's California Bearing Ratio (CBR).

Equivalency Factors

As might be inferred by the name, stabilized soils offer significant performance improvements over their previous non-stabilized origins. Through material testing, engineers realized that stabilization techniques enhanced the shear capacity of a soil significantly enough that stabilized soils outperformed traditional base and subbase materials (Ahlvin, 1991). They discovered that as the stabilization increases the bond between the soil aggregates, the soil begins to exhibit flexural strength similarly found in beams or rigid pavements. This improved flexural performance increases the stiffness of the respective material allowing the soil to better distribute wheel loads through the layer and reduce the vertical stress on the layer below, thus reducing the thickness required for the stabilized layer to mimic the performance of a conventional base or subbase course material (Mallick & El-Korchi, 2013). Realizing this benefit, engineers concluded they needed a method to account for the improved performance of stabilized soils (Ahlvin, 1991). As a general note, by increasing the stiffness of the stabilized layer, the tensile stress within the stabilized layer increases.

As the USACE was tackling this issue of stabilized soils in the 1970s, the empirical CBR method was the predominant method for designing flexible airfield pavements. With the empirical method in use, engineers decided to utilize equivalency factors to account for the added strength of stabilized soils; this technique was already in use by state highway officials but had not been considered for airfields (Ahlvin, 1991). Equivalency factors allow engineers to account for the incorporation of stabilized materials by reducing the thickness of the stabilized layer derived from the CBR method using traditional base or subbase course materials. This proportional relationship, as shown by numerical values in Table 2, provides engineers a ratio to determine the thickness of traditional base or subbase that can be substituted with an inch of a stabilized material. For example, a traditional subbase with a required thickness of 18 inches can be replaced by 9 inches of cement-stabilized, clayey gravel (GC).

Table 2. Equivalency Factors for Army and Air Force Pavements (U.S. Army Corps of Engineers, 2001)

Material	Equivalency Factors: Base	Equivalency Factors: Subbase
Asphalt-Stabilized		
All-Bituminous Concrete	1.15	2.30
GW, GP, GM, GC	1.00	2.00
SW, SP, SM, SC	--[1]	1.50
Cement-Stabilized		
GW, GP, SW, SP	1.15[2]	2.30
GC, GM	1.00[2]	2.00
ML, MH, CL, CH	--[1]	1.70
SC, SM	--[1]	1.50
Lime-Stabilized		
ML, MH, CL, CH	--[1]	1.00
SC, SM, GC, GM	--[1]	1.10
Lime, Cement, Fly Ash Stabilized		
ML, MH, CL, CH	--[1]	1.30
SC, SM, GC, GM	--[1]	1.40
Unbound Crushed Stone	1.00	2.00
Unbound Aggregate	–[1]	1.00

[1] Not used as base course.
[2] For Air Force Bases, cement is limited to 4 percent by weight or less.

The example above utilizes the approved equivalency factor for cement-stabilized, clayey gravel subbase per UFC 3-260-02; however, equivalency factors in use by other organizations depict this substitutive relationship to be too conservative. For example, with the Federal Aviation Administration (FAA) design standards, this same equivalency factor can be as high as 2.3 depending on the resulting resilient modulus (Federal Aviation Administration, 1995). If the true equivalency is closer to 2.3 than 2.0, it would mean the military is overdesigning airfield pavements by using estimates that are too conservative for this substitutive relationship.

The vast majority of research into the use of equivalency factors for airfields was undertaken in the 1970s by the USACE (Ahlvin, 1991; Sale, Hutchinson, Ulery, Ladd, & Barker, 1977). Typically, this research was conducted by analyzing one type of stabilized soil per experiment/report using full-scale, accelerated testing methods. These tests, as well as more recent full-scale tests conducted by the USACE, the FAA, and Airbus, are detailed further in Appendix H. Throughout the course of the literature review for this research, only one study was found that assessed the accuracy of equivalency factors using an aggregation of previous experimentation reports on the subject with a meta-analysis methodology. Sale et al. (1977) utilized three experiments to investigate cement, bituminous, and lime stabilization techniques; more information on their results is provided in Chapter III. In communications with the USACE and the AFCEC, the subject matter experts agreed that they had no recollection of any subsequent research specific to equivalency factors for flexible airfield pavements (Personal Communication, 11 Jan 2013). However, the literature review uncovered a number of reports that utilized aggregated historical test section data to increase the sample size of its experiments; these reports primarily dealt with other pavement topics, such as multiple-gear analysis (Barker & Gonzalez, 1994; Gonzalez, Barker, & Bianchini, 2012; Grau, 1973).

Problem Statement

As previously mentioned, subject matter experts at the AFCEC and their consultants have called into question the accuracy of the current equivalency factors for pavement design and evaluation. This reservation ultimately prompted this research effort with the overall goal to establish more realistic equivalency factors for design and evaluation of military airfields and to develop a procedure for the application of these factors. Although the primary focus is military airfields, the research could benefit non-military airfields as well.

Research Objectives

The overall research objective was to answer the question: how can the Air Force's current equivalency factors be adjusted to more accurately represent the actual structural capacity of stabilized layers by either developing new factors or adopting factors in-use by other organizations? This research objective was met by dissecting the larger objective into subsequent secondary objectives based upon the direction of the research sponsor, the AFCEC, and from the knowledge garnered from the literature review.

- Assess the accuracy of the Air Force's current equivalency factors using test section data from previous full-scale, accelerated pavement tests to evaluate the ability of the factors to accurately predict the structural capacity of stabilized soils in flexible pavement systems.

- Assess the accuracy of equivalency factors or methods used by other organizations to predict the structural capacity of stabilized soils using the previously mentioned test sections, and compare the predicative ability of these factors relative to the Air Force's equivalency factors.

- Compare the life-cycle costs of the various flexible pavement design methodologies using standard conventional and stabilized pavement designs.

- Determine the cost of utilizing stabilized soils in lieu of hauling conventional materials.

- Validate the Air Force's use of the Boussinesq-Frohlich Model for pavement systems with stabilized layers.

Research Approach

An extensive literature review was conducted to understand how different base and subbase materials react in response to different loadings, environmental conditions, thicknesses, and usages. From this initial review, the research then focused on identifying previously conducted laboratory and field tests pertaining to the study of equivalency factors and the effects of stabilized base and subbase materials. The results of the literature review helped identify the most suitable testing data for use in the analysis.

Using the full-scale testing data accumulated from the literature review, a meta-analysis was conducted to compare historical test section data from full-scale pavement tests using the actual thickness and the CBR-Beta (i.e., the USACE's current flexible airfield pavement design method) predicted thickness based on the failure coverages. Using these two thicknesses, an equivalency factor was computed for each test section. With the equivalency factors calculated, the different test sections were categorized by stabilized layer and method; a modeling simulation with one thousand trials was then used to expand the sample sizes for the different equivalency factors. After decomposing the equivalency factors into five percent increments from zero to one hundred percent, a sensitivity analysis was conducted to determine which equivalency factor value would result in the highest R^2 comparing the equivalent thickness of the test sections to the predicted thickness. The results of this analysis were then compared to the equivalency factors from the various federal and international aviation authorities (Objectives #1 and #2).

The research also investigated the life-cycle costs of the various flexible pavement design methods in use by the DoD and the FAA (Objective #3). This analysis was conducted by testing the various methodologies using standard design scenarios with both conventional and stabilized base courses. Using different thicknesses, an estimated initial construction cost was developed for each scenario. Combining this cost data with the reverse-calculated passes based on equivalent thickness using a standard design method (i.e., CBR-Alpha, the USACE's former design method), each design method was compared using a cost-per-pass metric. This metric allowed a comparison of each design method relative to the construction cost and service life. Converting the equivalent thickness to a predicted number of passes using a standard method was necessary to evaluate each method outside of its respective assumptions. Additionally, the cost data was used to determine the distance required for hauling conventional base course materials to equate to the additional costs of utilizing stabilization methods (Objective #4).

Based on suggestions by subject matter experts, the research sought to assess two of the assumptions inherent with the use of the CBR design method and equivalency factors: (1) the correlation between equivalency factors and shear capacity and (2) the use of the Boussinesq-Frohlich stress model to characterize the stress distribution through structural layers, particularly with respect to stabilized soils (Personal Communication, 2 Feb 2013). These two assumptions merged into the analysis of the concentration factor formulation based upon an analysis of the test sections with large variances between the equivalent thickness and the predicted thickness (Objective #5).

After completing the first five research objectives, the outputs from each objective were combined to formulate recommendations concerning equivalency factors for military airfields. This last portion of the research focused on fusing the results of the analysis and the recommendations into a useable product for design and evaluation of flexible pavements; this recommendation is presented in Chapter VII. Additionally, at the request of the sponsoring agency, AFCEC, the CBR-Beta's ability to model highway pavements was also assessed. The rationale behind this request was the ability to incorporate the vast amount of highway testing data into the empirical formulations for the airfield criteria. The graphical summarization of the research is presented as Figure 1.

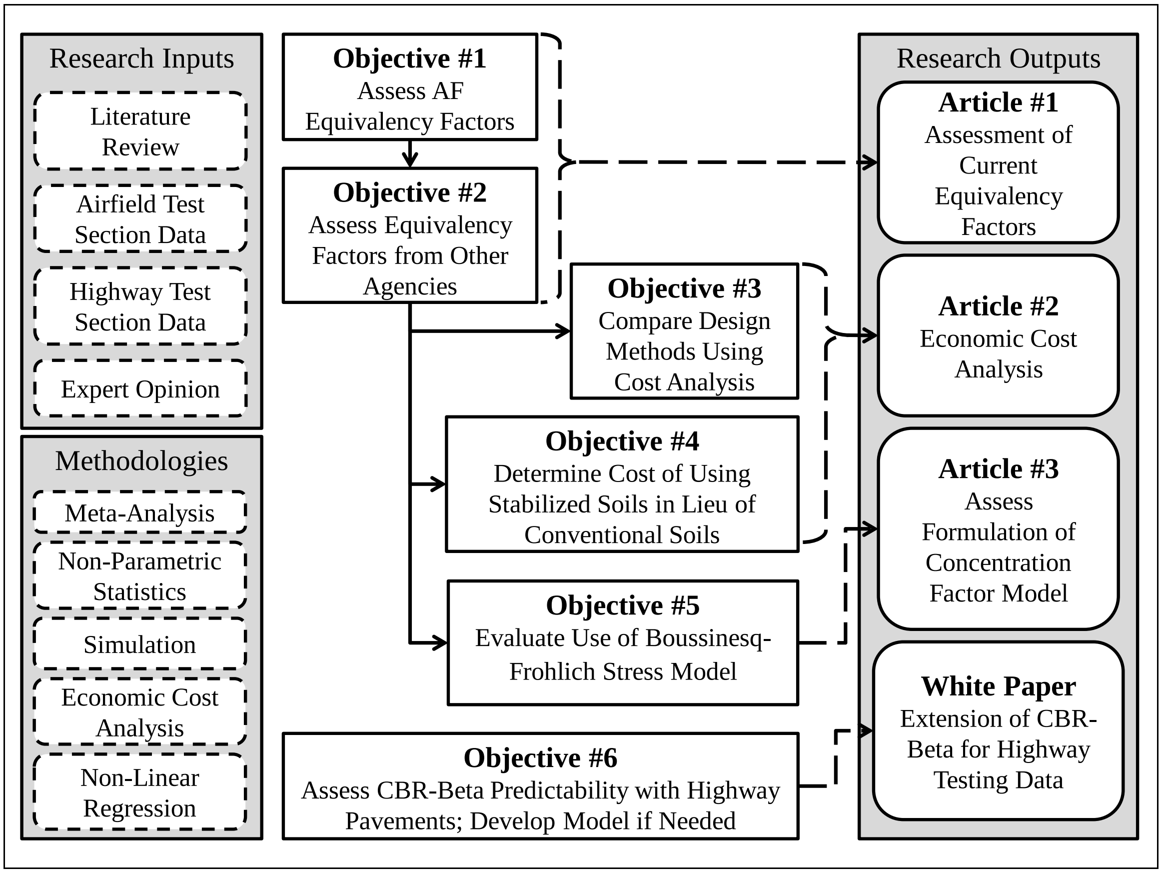

Figure 1. Summarized Research Approach

Scope and Limitations

As mentioned in greater detail in the research objectives and research approach sections of this chapter, this research focused on using a meta-analysis of historical test section data from various testing agencies to evaluate the Air Force's equivalency factors for flexible airfield pavements. Additionally, the test section data were used to evaluate the concentration factor formulation. Due to limitations with funding and laboratory facilities, this research effort relied solely on the analysis and documentation of previous pavement testing research efforts for data. An inherent limitation with utilizing this approach for data collection was that the analysis performed herein was dependent on representative testing data that accurately described soil behavior in real-world applications. This limitation was minimized by increasing the sample size with additional test samples where possible. For some equivalency factors though, available testing data were limited or non-existent.

Implications

The overall goal of the research effort was to establish more realistic equivalency factors for the design and evaluation of military airfields and to analyze the procedures for designing and evaluating stabilized soils. Based on the recommendations of this research effort, laboratory testing will be completed to verify the results by an Air Force civilian institute graduate student at the University of Cambridge. Upon verification, the recommendations will be included in an update to UFC 3-260-02 and UFC 3-260-03 for implementation; these revisions will be accomplished by the AFCEC and the USACE. Aviation authorities throughout the world rely on recommendations and research performed by the USACE; therefore, any recommendations to revising the current equivalency factors could potentially affect external organizations as well.

Preview

This chapter provided the necessary overview of the research topic to understand the problem, objectives, methodology, and the potential impacts. The remaining chapters of this thesis follow a scholarly article format with three separate articles and a white paper for submittal to peer-reviewed journals and the sponsoring agency; the last chapter, Chapter VII, provides a summary of the research, overall conclusions, and recommendations for future research. A literature review chapter applicable to all three journal articles was incorporated as Chapter II. Each of the three articles was written as a standalone document; however, the three articles tend to flow together with analysis from Chapters III and VI combining to develop a more in-depth and comprehensive analysis of the overall research question complete with recommendations for action. The first article (Chapter III) analyzed the Air Force's current equivalency factors and compares the predictability of these factors to other equivalency factors in use by other international, federal, and state agencies. The second article (Chapter IV) focused on evaluating the life-cycle costs of designing flexible airfield pavements using the various design methods. The third article (Chapter V) focused on analyzing the formulation of the concentration factor for use in design pavements and evaluating the vertical stress on the subgrade. A white paper (Chapter VI) assessed the use of the CBR-Beta design method to model highway pavements in an attempt to expand the database of historical test sections for empirical evaluation. Using the analysis and conclusions from the previous three articles and the white paper, the last chapter (Chapter VII) focused on fusing the findings together to provide recommendations and summarize the overall research effort.

II. Literature Review

The purpose of this chapter is to provide extended background information on the topics covered in subsequent thesis chapters. This chapter discusses flexible pavements, soil characteristics, the Boussinesq-Frohlich model, and the development of the current structural design methodology. Information related to the subject not contained in this chapter can be found in the literature reviews of the subsequent scholarly articles. In summary, this chapter establishes an introductory knowledge base about the subject for the three scholarly articles and the white paper.

Flexible Pavements

Flexible pavements are used in several different applications throughout the world, but for the purposes of this research only airfield applications are discussed. In airfield applications, flexible pavement systems are thicker and more expensive to construct and maintain than other flexible pavements due primarily to the characterization of the loads, which includes high-pressure tires and heavy wheel loads. These special cases can include tire pressures as high as 350 pounds per square inch (psi) and aircraft loads over 800 thousand pounds (U.S. Army Corps of Engineers, 2001). As a general note, these two conditions typically do not occur simultaneously as aircraft manufacturers design heavier aircraft with more complex landing gear configurations to dissipate the load and reduce contact pressures on the pavement.

As Figure 2 suggests, asphalt (or flexible) pavements consist of built-up structural layers that carry and distribute loads to the underlying layer in an overall effort to support the vehicle load on the wearing course. As discussed in further detail later, wheel loads are most severe on the surface at the point of contact and dissipate as the load travels downward through the structural layer; this logic follows that the strongest and least flexible granular layers are located nearest the surface with strength decreasing with each layer as depth increases and stress decreases. The general rationale is the stronger layer is designed, based upon thickness and material characteristics, to support the stresses from the layer above and distribute the load through the respective layer to the structural layer below at distributed stress levels the lower layer can support.

Soil Characteristics

Engineers use several different variables to characterize the soils that comprise the base, subbase, and subgrade courses to include soil classification, resilient modulus, California Bearing Ratio (CBR), and Poisson's Ratio. These variables are not the only soil characteristics; however, in terms of pavement design, these variables are the most frequently used. This section introduces each of these soil characteristics.

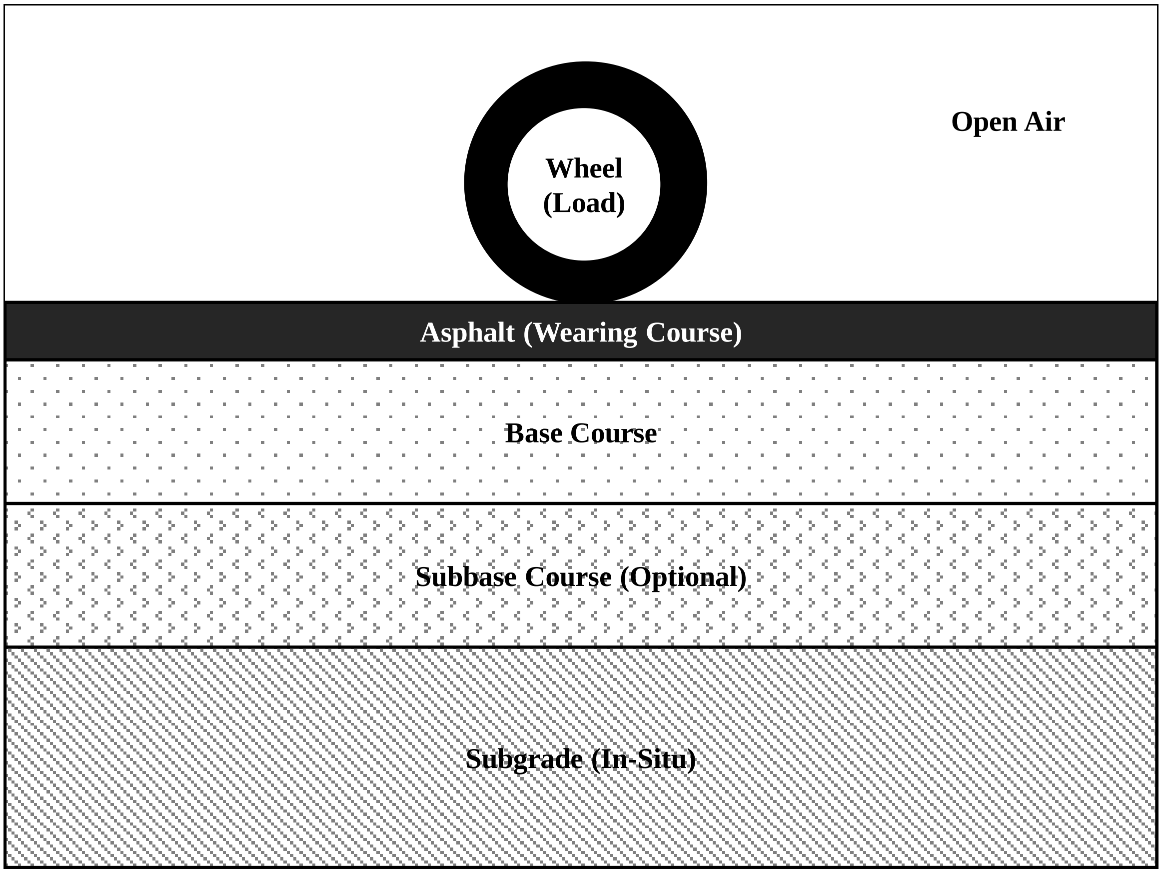

Figure 2. Typical Flexible Pavement Structure

Soil Classification

Das (2005) identified the different systems that exist to classify soil; however, the Unified Soil Classification System (USCS) is the primary soil classification system for military engineering, and is widely used outside of the military as well. Dr. Arthur Casagrande developed the USCS for the U.S. Army Corps of Engineers (USACE) in 1942 specifically for large-scale, airfield construction effort undertaken by USACE during World War II. The U.S. Bureau of Reclamation later revised the USCS in 1952; the American Society for Testing and Materials (ASTM) later accepted the revised USCS as a universally approved soil classification method (Das, 2005).

The USCS classifies soil into three broad categories: coarse-grained, fine-grained, and highly organic soils (shown in Table 3). Of these three categories, only coarse-grained and fine-grained soils are further subdivided, as highly organic soils are not suitable for use in construction to include airfield pavements. U.S. Army Field Manual (FM) 5-530 fully describes the USCS and details each of the further subdivisions; summarizes the divisions and provides pertinent information about each soil type necessary for airfield construction considerations. For flexible pavements, most base course materials are well-graded gravels (GW) or more typically crushed stone; however, the layers below the base course can potentially be any combination of other soil types depending on the local area, provided they meet strength and gradation requirements.

Resilient Modulus

According to the ASTM, the resilient modulus of a soil indicates the stiffness of the material which is then used to approximate in-situ response (Durham, Marr, & DeGroff, 2003). This material property is similar to the elastic modulus used for other materials, such as steel, in that the resilient modulus is a measure of the material's ability to resist permanent deformation after loading. The primary difference is the resilient modulus accounts for the repeated loading of the material; soils, particularly those under traffic loads, do not experience the same type of loadings that other materials experience. Additionally, soils do not fully exhibit the elastic properties of other materials as repeated loads typically cause permanent deformation. Research conducted in the 1960s and 1970s further support this statement as it was determined the behavior of soils under traffic loading could be assessed only from repeated load tests, and this property was best

Table 3. Unified Soil Classification System (Recreated from U.S. Army Engineer School, 1987)

Major Divisions			USCS Classification	Description	Value as Base Course Material	Compressibility and Expansion	Drainage Characteristics	Dry Unit Weight (pcf)	CBR
Course-Grained Soils	Gravels	<5% Fines	GW	Well-graded gravels or gravel-sand mixtures, little or no fines	Good	Almost None	Excellent	125-140	60-80
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines	Poor to Fair	Almost None	Excellent	110-130	25-60
		>12% Fines	GM	Silty gravels, gravel-sand-silt mixtures	Fair to Good (Poor when LL>28)	Very Slight to Slight	Fair to Poor (Impervious when LL>28)	120-145	20-80
			GC	Clayey gravels, gravel-sand-clay mixtures	Poor	Slight	Poor to Practically	120-140	20-40
	Sands	<5% Fines	SW	Well-graded sands or gravelly sands, little or no fines	Poor	Almost None	Excellent	110-130	20-40
			SP	Poorly graded sands or gravelly sands, little or no fines	Poor to Not Suitable	Almost None	Excellent	100-120	10-25
		>12% Fines	SM	Silty sands, sand-silt mixtures	Poor to Not Suitable	Very Slight to Medium	Fair to Poor (Impervious when LL>28)	105-130	10-40
			SC	Clayey sands, sand-clay mixtures	Not Suitable	Slight to Medium	Poor to Practically	105-130	10-20
Fine-Grained Soils	Silts & Clays (Liquid Limit <50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasicity	Not Suitable	Slight to Medium	Fair to Poor	100-125	5-15
			CL	Inorganic clays of low to medium plasiticity, gravelly clays, sandy clays, silty clays, lean clays	Not Suitable	Medium	Practically Impervious	100-125	5-15
			OL	Organic silts and organic silt-clays of low plasiticity	Not Suitable	Medium to High	Poor	90-105	4-8
	Silts & Clays (Liquid Limit >50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils,	Not Suitable	High	Fair to Poor	80-100	4-8
			CH	Inorganic clays of high plasiticity, fat clays	Not Suitable	High	Practically Impervious	90-110	3-5
			OH	Organic clays of medium to high plasicity, organic silts	Not Suitable	High	Practically Impervious	80-105	3-5
Highly Organic Soils			Pt	Peat and other highly organic soils	Not Suitable	Very High	Fair to Poor	N/A	N/A

characterized by the use of the resilient modulus (Groeger, Rada, & Lopez, 2003). It is worth noting that the resilient modulus (M_r) value for a particular material measures the stiffness under repeated loadings at different stress levels; this characteristic is represented by Equation 1:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{1}$$

where σ_d is the deviator stress and ε_r is the recoverable strain.

Although there is debate over how to most accurately measure the resilient modulus, organizations such as the Federal Highway Administration (FHWA), the Federal Aviation Administration (FAA), and the Department of Defense (DoD) have adopted the material property as a primary performance indicator of granular materials for pavements (Groeger, Rada, & Lopez, 2003). This statement is particularly true for agencies that use layered-elastic theory to design pavement systems. Currently, resilient modulus research is attempting to address how best to measure the property and if laboratory testing is representative of in-situ performance (Durham, Marr, & DeGroff, 2003).

California Bearing Ratio (CBR)

The U.S. military adopted the CBR test in the mid 1940s, after extensive field validation tests, as a method of characterizing the strength of a given soil; the CBR thus became the basis of the military's flexible pavement design methodology (Ahlvin, 1991). The CBR characterizes soil strengths based upon a respective soil's strength relative to the strength of crushed limestone as a percentage, which has a CBR of 100. The methodology follows that the stronger a material, the higher it will rate relative to the crushed limestone; conversely, weak materials, such as fat clays (CH), rate on the low end of the rating scale at or below five percent. Figure 3 graphically depicts the CBR scale; the USCS classified soils are included for comparison.

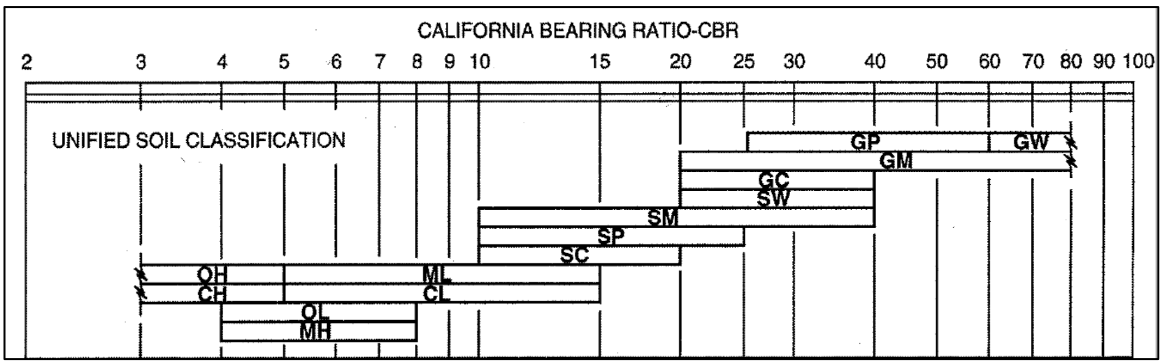

Figure 3. Relation of CBR to USCS (U.S. Army Corps of Engineers, 2001)

As previously stated, the structural layers in a pavement system are oriented with the strongest materials closest to the surface. This statement is further supported by the excerpt shown in Table 4, which summarizes the guidelines in the Unified Facility Criteria (UFC) 3-260-02, *Pavement Design for Airfields*. Soils in the subbase, for example, can exceed 50; however, these soils would then be required to meet the gradation and material properties for base course materials (U.S. Army Corps of Engineers, 2001). This event is unlikely as all uniformed services have established minimum thickness requirements for the wearing and base courses to ensure the stresses from the wheel loads are distributed to a level such that the subbase materials are not required to exceed a CBR of 50. In doing so, the DoD ensured that cheaper local soils would typically be used as subbase materials in lieu of hauling more engineered soils a potentially great distance.

Table 4. Typical CBR Values for Different Material Layers (U.S. Army Corps of Engineers, 2001)

Material Layer	CBR Range
Base Course	80-100
Subbase Course	20-50
Subgrade Course	In-Situ Soil (Typically <20)

Poisson's Ratio

Poisson's ratio is a measure of the Poisson effect for a given shape. This effect holds that when a three-dimensional shape is compressed in the axial direction, it will compress in the axial direction and expand in the transverse direction; this statement is conversely true for shapes in axial tension. Soils experience axial compression along the vertical axis when loaded causing individual soil elements (in terms of infinitesimally small cubic elements) to compress; this compression along the vertical axis causes an expansion of the element along the transverse axis. The axial compression from a wheel load on soil is a vertical deformation immediately under the wheel and a transverse movement of the displaced soil away from the wheel. The cumulative effect of this action across the soil elements shows on the surface in the form of surface rutting and upheaval. The Poisson's ratio (v) for a given soil is difficult to measure in a laboratory and has a relatively minor influence on strength compared to other material properties; therefore, subject matter experts recommend using standard values as shown in Table 5.

Table 5. Typical Poisson's Ratios for Four Classes of Pavement Materials (U.S. Army Corps of Engineers, 2001)

Pavement Materials	Poisson's Ratio v
Bituminous concrete	0.5 for E < 3,450 MPa (500,000 psi) 0.3 for E > 3,450 MPa (500,000 psi)
Unbound granular base- or subbase-course	0.3
Chemically stabilized base- or subbase-course	0.2
Subgrade	
Cohesive subgrade	0.4
Cohesionless subgrade	0.3

Note: E = elastic modulus of bituminous concrete (psi)

Boussinesq-Frohlich Model

As basic engineering principles would suggest, the compressive (axial) stress on an object is equivalent to the load divided by the area. This principle holds true in many different applications; however, it was not until Joseph Boussinesq developed his stress distribution in 1883 that engineers understood how to apply this concept to soil mechanics (Das, 2005). Boussinesq understood that in soils, the stress distribution is affected by depth because soils, when properly compacted, demonstrate an "arching" effect similar to that found in masonry. This arching effect varies by soil classification, but it holds that the more dense and strong the material, the more it behaves like a flexural member; therefore, dense materials have higher angles of dispersion and greater load distributing properties. Typically most granular soils have dispersion angles around 45 degrees, with fine grained soils closer to 25 degrees. The angle of dispersion is important as it accounts for the increase in loaded area (the area increases along the dispersion angle as depth increases). When factored into the basic compressive stress relationship, this characterization of the angle of dispersion implies that with added depth, the vertical stress in the soil decreases as depth increases since the load is distributed over a larger area.

In his initial work, Boussinesq did not account for the effects of different soils on load distribution; it was not until O.K. Frohlich incorporated his concentration factor into the Boussinesq formulation that the characteristics of the different soils were considered. Boussinesq developed his stress distribution model based upon this arching characteristic. The Boussinesq model was used by geotechnical and transportation engineers, as originally developed, until the 1930s when Frohlich reformulated the model and incorporated a concentration factor (Olmstead & Fischer, 2009). Frohlich's concentration factor is an empirically derived factor designed to more closely align the computed stresses with laboratory-measured stresses. In its current form (Equation 2), the Boussinesq-Frohlich stress equation is still widely used to calculate the vertical stress at an arbitrary point under a point load (Gonzalez, Barker, & Bianchini, 2012):

$$\sigma_z = \frac{nP}{2\pi R^2} cos^n(\theta)) \qquad (2)$$

where,

P = Point Load Applied at the Surface
σ_z = Vertical Stress at an Arbitrary Point
R = Distance from the Point Load to the Location of Interest
Θ = Angle Between the Vertical Axis and the Line Connecting the Point Load to the Location of Interest
n = Frohlich's Concentration Factor

Development of the Structural Design Methodology

In the early 1940s, USACE assumed responsibility for design and construction of military airfields; at the time, the Air Force was still the U.S. Army Air Corps (Ahlvin, 1991). As World War II started and the necessity for heavy bombers became evident, USACE realized a pavement design method was necessary to support the heavier loads of the aircraft at the time. It looked at several promising methods for pavement design, which all relied on the bearing capacity of the subgrade; however, at the time, USACE did not have a suitable method for characterizing the bearing capacity of the subgrade, particularly in contingency environments. The subject matter experts at USACE realized a rational method for design was necessary to limit the stress and strain on the subgrade soil, but due to the time constraints imposed by the conflicts overseas conceded that an empirical method was more prudent (Ahlvin, 1991). As previously mentioned, USACE ultimately adopted the CBR method for characterizing the strength of the soils in a pavement system, which ultimately led to the establishment of the CBR design methodology for flexible airfield pavements.

USACE conducted full-scale, accelerated pavement testing in the 1940s to modify the empirical CBR design method, used by state highway officials for roadway design, for airfield use (Gonzalez, Barker, & Bianchini, 2012). The early full-scale tests helped address the differences between aircraft and highway loadings; these tests specifically addressed heavy single wheels, effect of dual wheels, and the effect of high-pressure tires on pavements (Ahlvin, 1991). With a limited experience in design and construction, USACE developed the initial CBR design equation (Equation 3) for flexible airfield pavements (Ahlvin, 1991):

$$t = k\sqrt{P} \tag{3}$$

where,

t = Thickness of Pavement Structure
P = Wheel Load
k = Design Constant for a Particular CBR and Tire Pressure

Subsequent testing in the decades to come, and particularly in the 1970s, further refined the initial CBR equation. These later tests began to incorporate experimentations with larger cargo aircraft, multiple wheel configurations, mixed traffic, design for different coverage levels and airfield surfaces (such as runway, apron, and taxiway), and stabilized soils (Ahlvin, 1991). Of the tests conducted by USACE in the 1970s, the most prominent one was the Multi-Wheel Heavy Gear Load (MWHGL) Test, which evaluated the impact of heavy cargo aircraft, weighing over a half-million pounds, on airfield pavements. This test represents one of the last major full-scale tests conducted by USACE and is among the most referenced tests concerning flexible airfield pavement design (Gonzalez, Barker, & Bianchini, 2012). The MWHGL tests resulted in the most significant change to the CBR design methodology with the revised form of the CBR design equation (CBR-Alpha) presented as Equation 4 (Gonzalez, Barker, & Bianchini, 2012):

$$t = \alpha\sqrt{\frac{ESWL}{8.1*CBR} - \frac{A}{\pi}} \tag{4}$$

where,

t = Thickness of Pavement Structure

α = Load Adjustment Factor (Function of Traffic Volume and Number of Tires)
ESWL = Equivalent Single Wheel Load
A = Tire Contact Area

This formulation is based upon three design concepts. First, each structural layer must be thick enough to distribute loads through the depth of the layer resulting in a stress level that does not overstress and produce deformation in the layer below (Ahlvin, 1991). This requirement dictates the thickness of each structural layer based upon the soil properties and the load case. Additionally, this requirement drives the necessity for minimum thickness requirements for each layer as a method of ensuring each sub layer is protected sufficiently by the layer above. Second, when constructing pavement systems, each structural layer must be sufficiently compacted to ensure that aircraft loading does not produce an unintended compactive effort (Ahlvin, 1991). Without proper compaction, flexible pavements will fail prematurely from unserviceable levels of rutting. Third, flexible pavements must have a wearing course of some medium to protect the structural layers that will not displace under load (Ahlvin, 1991).

For ease of use, the CBR method converts the design equation into design curves. The design curves graphically determine the required thickness above the subgrade based upon the subgrade CBR, aircraft type, aircraft gross weight, and the required number of passes. Figure 4 depicts a design curve for F-15 aircraft as given in UFC 3-260-02. USACE created similar charts for pavement evaluation.

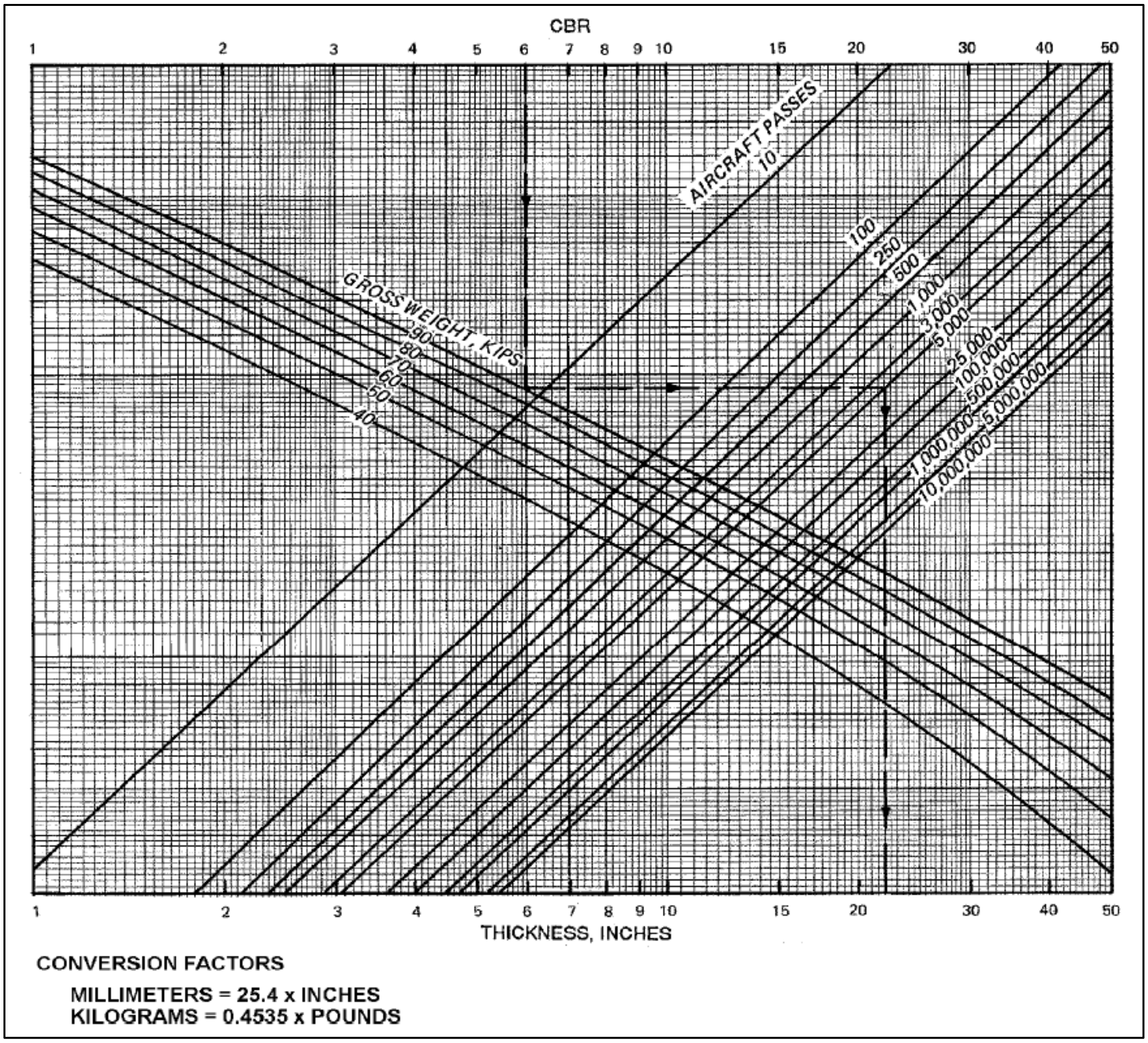

Figure 4. Flexible Pavement Design Curve for F-15, Type A Traffic Areas (U.S. Army Corps of Engineers, 2001)

CBR-Beta Methodology

Recognizing the largely empirical nature of the CBR-Alpha design methodology (Equation 4) and its inability to accurately model the loads associated with newer, heavier aircraft, such as the Boeing 777, a USACE research team, in the late 2000s, developed the CBR-Beta methodology, which successfully transitioned the CBR design method from a strictly empirical model to a mechanistic-empirical method. The research team asserted that the inclusion of the Boussinesq-Frohlich stress distribution model resulted in a model that calculated soil stress; the resulting stress values were related to pavement performance using historical traffic test data (Gonzalez, Barker, & Bianchini, 2012). Starting with the original CBR model (Equation 3) and the Boussinesq-Frohlich model (Equation 2), the research team, through mathematical derivation, fused the two models into one design equation. The results of their efforts are shown as Equation 5:

$$\frac{t}{r} = \frac{1}{\sqrt{\left(\frac{1}{1-\frac{\beta CBR}{\pi \rho}}\right)^{\frac{2}{n}} - 1}} \tag{5}$$

where,

t = Thickness of Pavement Structure
r = Loaded Radius
β = Beta Factor (Function of Coverages and Stress)
CBR = California Bearing Ratio
ρ = Contact Pressure
n = Stress Concentration Factor, ($n = 2\left[\frac{CBR}{6}\right]^{0.1912}$)

During the derivation and subsequent acceptance testing of the equation, the research team concluded that the most accurate method to calculate the subgrade stresses was to derive the concentration factor as a function of subgrade CBR; this was done by analyzing single-wheel test section data from the Stockton Airfield Tests conducted in the late 1940s. When applied to typical subgrade CBR values, the modified factor ranges from 1.75 to 2.38 for CBR values of 3 to 15, respectively. For comparison, the research team concluded that CBR-Alpha criteria unknowingly followed a stress concentration factor of 2.0. Using this value, the subgrade stresses were routinely over-estimated for low-strength subgrades and under-estimated for high-strength subgrades. The research team concluded that the CBR-Alpha criteria resulted in both over- and under-designed pavements as shown in Figure 5 (Gonzalez, Barker, & Bianchini, 2012).

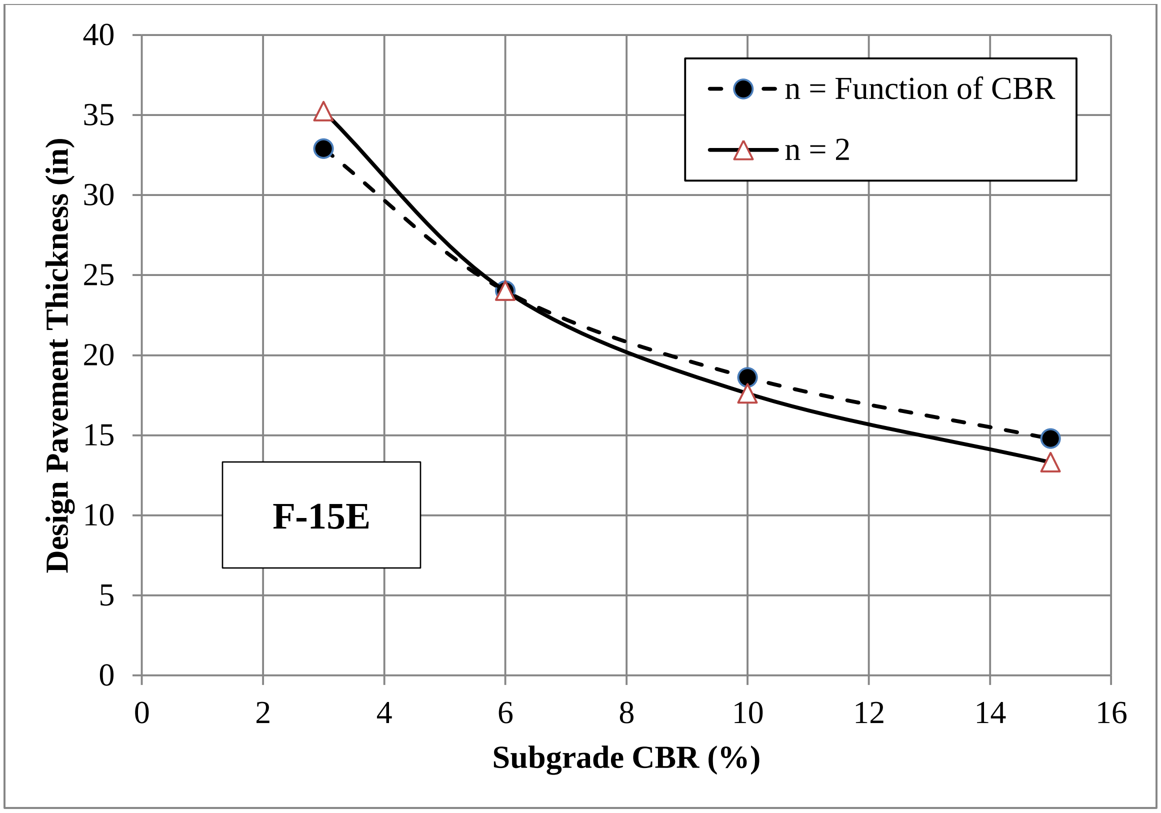

Figure 5. Design Curves for F-15 using *n* as Function of CBR (Recreated with Current Variable Formulations from Gonzalez, Barker, & Bianchini, 2012)

Layered Elastic Methodology

In 1975, USACE developed its layered-elastic design method based upon Donald Burmeister's layered theory (Ahlvin, 1991). At the time, USACE presented this design methodology as an optional method for flexible pavements; however, guidance from

UFC 3-260-02 now recommends the use of this method as a primary method for stateside locations. Building upon the theory and methodology from the USACE work, the FAA developed its own layered-elastic design method in the mid 1990s known as LEDFAA (Brill, 2012a). After seeing success with LEDFAA, the FAA further refined its layered-elastic design method with the release of FAA Rigid and Flexible Iterative Elastic Layered Design (FAARFIELD) in 2009. With the release of FAARFIELD, the FAA stopped using the CBR method for design (Federal Aviation Administration, 2009).

Overall, layered-elastic methods are largely mechanistic and based extensively on layered theory, which represents an evolution of the Boussinesq-Frohlich stress distributions with the added benefit of including the ability to represent the varying stiffness of different materials and the interaction between the layers in a pavement system. In doing so, these methods rely on estimating or knowing the material properties of a given layer. In a contingency environment, these material properties are often hard to test or estimate; therefore, the DoD still relies on the CBR method, particularly with pavement evaluation as it is difficult to characterize the degradation of a material over time with layered-elastic methods.

Summary

This chapter provided additional background information on the topics covered in subsequent chapters. The chapter discussed flexible pavements, soil characteristics, the Boussinesq-Frohlich model, and the development of the current structural design methodology. Information related to the subject not contained in this chapter can be found in the literature reviews of the subsequent scholarly articles. In summary, this chapter established an introductory knowledge base about the subject for the three scholarly articles and the white paper.

III. Journal Article: Investigation of Equivalency Factors for Flexible Airfield Pavements

The journal article presented in this chapter is intended for submission to the *Transportation Research Record: Journal of the Transportation Research Board (TRB)*. The journal article presents the results of the meta-analysis performed to develop more representative equivalency factors for the design and evaluation of flexible airfield pavements. While the content of this chapter is the same as the journal submission, formatting adaptations have occurred for inclusion in this thesis. Further support and information regarding the content contained in this article are available in Appendix A.

Investigation of Equivalency Factors for Flexible Airfield Pavements

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Abstract

In an effort to address the use of equivalency factors in flexible airfield pavement design, this research conducted a meta-analysis of historical full-scale, accelerated airfield pavement tests to assess the accuracy of equivalency factors in characterizing the additional strength provided by stabilized soils. An experimental equivalency factor was calculated for each test section using the U.S. Army Corps of Engineers' (USACE) California Bearing Ratio-Beta (CBR-Beta) design methodology. The test section data was then segregated based upon the stabilization method and layer (i.e., base and subbase course). Each of the groupings was analyzed using non-parametric statistics, simulation analysis, and optimization to determine the more representative equivalency factor for a given stabilized soil for a particular base or subbase course. Ultimately, this analysis led to the revision of eight of the USACE's equivalency factors.

Key Words (Subject Headings)

Equivalency Factors; Flexible Pavements; Military Airfields; Pavement Design; Stabilized Soils; CBR-Beta

Introduction

Costing approximately $213 million (2013 dollars), the American Association of State Highway Officials (AASHO) road test represents possibly the largest civil engineering experiment ever completed in the United States (Fenves, Fisher, & Viest, 2005). Starting in 1956 and lasting five years, the AASHO road test built the foundation for highway pavement design worldwide; the results from the test are widely used to this day (Hudson, Monismith, Shook, Finn, & Skok, 2007). Of the many breakthroughs found during the field-testing, the AASHO road test demonstrated that stabilized soils offer significant load distributing improvements. The AASHO road test enabled engineers to use equivalency factors on highway work.

Following this revelation with stabilized soils for highway work, the U.S. Army Corps of Engineers (USACE) began its own experiments with stabilized soils; this line of research focused on airfield applications. By 1974, the USACE conducted several full-scale pavement tests; these tests resulted in the acceptance of stabilized soils for airfield pavements. In 1977, the USACE developed its own equivalency factors for the design and evaluation of flexible airfield pavements from analyzing the results of its full-scale experiments (Ahlvin, 1991).

Since the USACE first published its equivalency factors in the 1970s, several other airfield pavement authorities, to include the Federal Aviation Administration (FAA), have developed equivalency factors. As with most factors derived through empirical means, debate exists among the different airfield pavement authorities as to the most accurate method to characterize stabilized soils. Reviewing their organization's equivalency factors, subject matter experts at the USACE and the Air Force Civil

Engineer Center (AFCEC) familiar with the development and utilization of these factors have raised concerns about the accuracy and application of the factors (Personal Communication, 3 Jan 2013). Further supporting this sentiment, in 2010, the USACE contracted a team of experts to evaluate its pavement program in its entirety. The experts made several recommendations to the USACE to include a reevaluation of equivalency factors as they believed the current factors were overly conservative and did not accurately characterize the superior performance of stabilized soils (Monismith, Thompson, Leahy, & Zapata, 2010). In its concluding remarks, the team listed equivalency factors among its top recommendations to the USACE for corrective action.

Objectives

As previously mentioned, subject matter experts at the USACE and their consultants have called into question the accuracy of the current equivalency factors for pavement design and evaluation. This uncertainty ultimately prompted this research effort with the overall goal to establish more realistic equivalency factors for design and evaluation of military airfields and to develop a procedure for the application of these factors. Although the primary focus was military airfields, the research can benefit non-military airfields as well.

This article represents a small portion of a larger research effort to revise the Army and Air Force's equivalency factors; the two services use the same set of equivalency factors. The research specifically assessed the accuracy of equivalency factors to predict structural capacity of stabilized soils in flexible pavement systems using full-scale accelerated pavement test sections. The research also evaluated the predictability of the Army and Air Force's equivalency factors relative to the predictability of factors from other state, federal, and international pavement authorities.

Methodology

Throughout the course of the literature review for this research, only one study was found that assessed the accuracy of equivalency factors using an aggregation of previous experimentation reports on the subject with a meta-analysis methodology. Sale, Hutchinson, Ulery, Ladd, and Barker (1977) utilized three experiments to investigate cement, bituminous, and lime stabilization techniques. From their report, the researchers provided four conclusions for flexible pavements:

- Equivalency factors should be bounded between 1.0 and 2.3.
- An equivalency factor used in the base course can be multiplied by two to be applied in the subbase course. This follows the rationale that the base course, in terms of CBR, is at least twice the strength of the subbase; therefore, a stabilized soil adequate for the replacement of base course material would provide twice the benefit if used in the subbase course.
- The researchers recommended that the equivalency factors for bituminous stabilized soils be used as a point estimate. These recommended equivalency factors are still used today and are listed under the asphalt-stabilized heading in Table 6 [current equivalency factors as published in Unified Facilities Criteria (UFC) 3-260-02, *Pavement Design for Airfields*].
- Conversely, the researchers concluded that the equivalency factors for lime, cement, or a combination of lime, cement, and fly ash are calculated as a function of the unconfined compressive strength of the stabilized material.

The conclusions from Sale et al.'s (1977) research ultimately served as the foundation for the Army and Air Force's current set of equivalency factors. Comparing their conclusions from the report with the current equivalency factors shown in Table 6, it is apparent that little has changed from 1977 to the present day. As a general note, equivalency factors are entirely empirical and are specific to the design method and assumptions under which they were derived; this includes variations in equivalent thickness determination. The current set of equivalency factors were derived using the California Bearing Ratio (CBR) Alpha design method implying that the current set of equivalency factors are not necessarily calibrated to the new CBR-Beta design method. For general reference, the CBR-Alpha method was the USACE's primary flexible airfield design method from the 1970s to the late 2000s, when it was gradually phased out in favor of the new mechanistic-empirical CBR-Beta (Gonzalez, Barker, & Bianchini, 2012).

Table 6. Equivalency Factors for Army and Air Force Pavements (U.S. Army Corps of Engineers, 2001)

Material	Equivalency Factors: Base	Subbase
Asphalt-Stabilized		
All-Bituminous Concrete	1.15	2.30
GW, GP, GM, GC	1.00	2.00
SW, SP, SM, SC	--[1]	1.50
Cement-Stabilized		
GW, GP, SW, SP	1.15[2]	2.30
GC, GM	1.00[2]	2.00
ML, MH, CL, CH	--[1]	1.70
SC, SM	--[1]	1.50
Lime-Stabilized		
ML, MH, CL, CH	--[1]	1.00
SC, SM, GC, GM	--[1]	1.10
Lime, Cement, Fly Ash Stabilized		
ML, MH, CL, CH	--[1]	1.30
SC, SM, GC, GM	--[1]	1.40
Unbound Crushed Stone	1.00	2.00
Unbound Aggregate	–[1]	1.00

[1] Not used as base course.
[2] For Air Force Bases, cement is limited to 4 percent by weight or less.

In communications with USACE and the AFCEC, the subject matter experts agreed that they had no recollection of any subsequent research specific to equivalency factors for flexible airfield pavements (Personal Communication, 11 Jan 2013). However, the literature review uncovered a number of reports that utilized aggregated historical test section data to increase the sample size of experiments; these reports primarily dealt with other pavement topics, such as multiple-gear analysis (Barker & Gonzalez, 1994; Gonzalez, Barker, & Bianchini, 2012; Grau, 1973). The meta-analysis methodology used in these previous pavement research efforts formed the basis of the current analysis for studying equivalency factors.

Using the full-scale testing data accumulated from the literature review, a meta-analysis was conducted to compare historical test section data from full-scale pavement tests using the predicted thickness, based on the number of coverages at failure using the USACE's CBR-Beta design method, and the actual thickness. The CBR-Beta design equation is shown in Equation 6:

$$\frac{t}{r} = \frac{1}{\sqrt{\left(\frac{1}{1-\frac{\beta CBR}{\pi \rho}}\right)^{\frac{2}{n}} - 1}} \tag{6}$$

where,

t = Thickness of Above Subgrade
r = Contact Radius
β = Beta Factor (Function of Coverages and Stress)
CBR = California Bearing Ratio
ρ = Contact Pressure (Equivalent Pressure for Multiple-Wheel Gear Assemblies)
n = Stress Concentration Factor,
($n = 2\left[\frac{CBR}{6}\right]^{0.1912}$)

The CBR-Beta process utilizes the formulation provided in Equation 6 for single-wheel loads and as the initial prediction for multi-wheel gear assemblies. For multi-wheel gear assemblies, the CBR-Beta process requires that the vertical stress from each wheel be analyzed separately and then superimposed for each analysis point to be evaluated below the assembly. Using the superimposed stresses, the depth of the subgrade is then increased, as necessary, to reduce the vertical stress until the stress is equal to the allowable stress for the given test section as defined in Equation 7:

$$\sigma_{allowable} = \frac{\beta CBR}{\pi} \quad (7)$$

Using the predicted and actual thicknesses values, an experimental equivalency factor was calculated using the USACE's equivalent thickness process shown in Figure 6. The equivalent thickness process uses the actual layer thicknesses combined with the necessary equivalency factors to determine the equivalent thickness of conventional soil above the subgrade; this process is necessary due to the homogenous layer assumption of the CBR and Boussinesq methods. To determine an equivalency factor, the equivalent thickness formulation was set equal to the CBR-Beta predicted thickness. Using the actual layer thicknesses and mathematical manipulation, the equivalent thickness formulation was solved in terms of the equivalency factor of interest.

Aviation authorities typically tend to calculate equivalent thickness slightly differently; however, as these factors are designed for use with the CBR-Beta method, the USACE's equivalent thickness method was used. An important distinction between the USACE method and the FAA method is that the USACE method does not subtract an equivalent amount of subbase when the base or the wearing course do not meet minimum thickness requirements (Personal Communication, 4 Feb 2014).

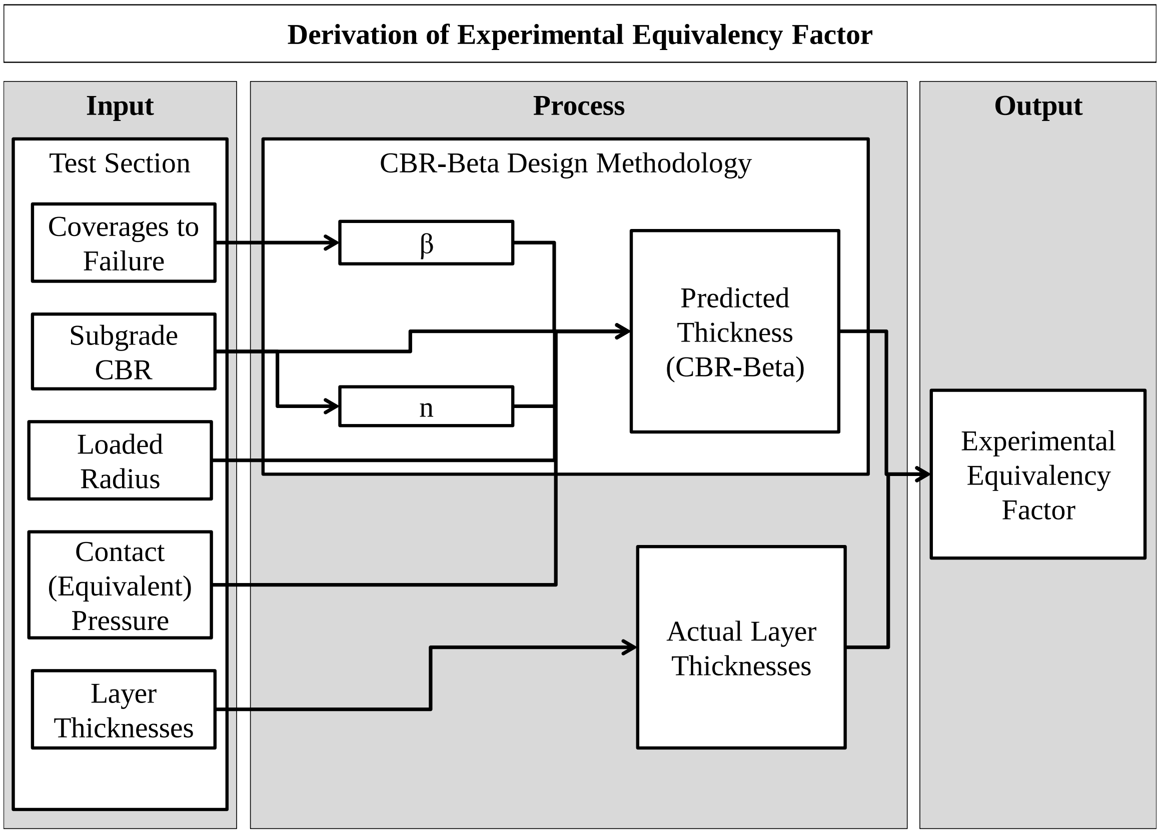

Figure 6. Overview of Test Section Data Conversion for Analysis

As shown in Table 6, the Army and the Air Force currently use 18 different equivalency factors: 5 for stabilized base course materials and 13 for stabilized subbase course materials (U.S. Army Corps of Engineers, 2001). To improve the statistical confidence of the analysis, at least 30 representative samples for each of the equivalency factors was preferred; however, such a quantity of data simply does not exist for any of the equivalency factors. As a result, the calculated equivalency factors from the test sections were extrapolated using a simulation of 1,000 trials each; the simulation was analyzed using nonparametric statistics to compare the simulation to the actual data.

Simulation using Triangular Distributions

When analyzing the historical data, the variables necessary to calculate the experimental equivalency factor (i.e., contact radius, contact pressure, subgrade CBR, failure coverages, and layer thicknesses) appeared, for the vast majority of the input variables, to follow a triangular distribution. This realization prompted the idea that a simulation could help increase the statistical confidence of the analysis by increasing the small samples for each equivalency factor to large samples with 1,000 data points each. A triangle distribution was established for each of the independent variables, where applicable, for a respective stabilization method and layer type using its respective minimum, maximum, and median values. The distributions for a select number of subbase input variables were modeled as continuous uniform, due to the small sample size, rather than being discretized or forced to fit a triangular distribution.

A random number between zero and one was used to calculate a random value for each variable based upon the cumulative distribution created using the minimum, maximum, and median values of the actual data. Using a correlation matrix that included only the input variables for a respective equivalency factor, the input variables were grouped according to their influence on either the predicted or actual thickness. Positive correlations were paired together and associated with a random number between one and zero. Similarly, the negative correlations were paired and assigned the complementary random number used for the positive correlations. In a single instance for a subbase material, an input variable had zero correlation; therefore, a unique random variable was used. Figure 7 summarizes the process of creating random variables for the simulation.

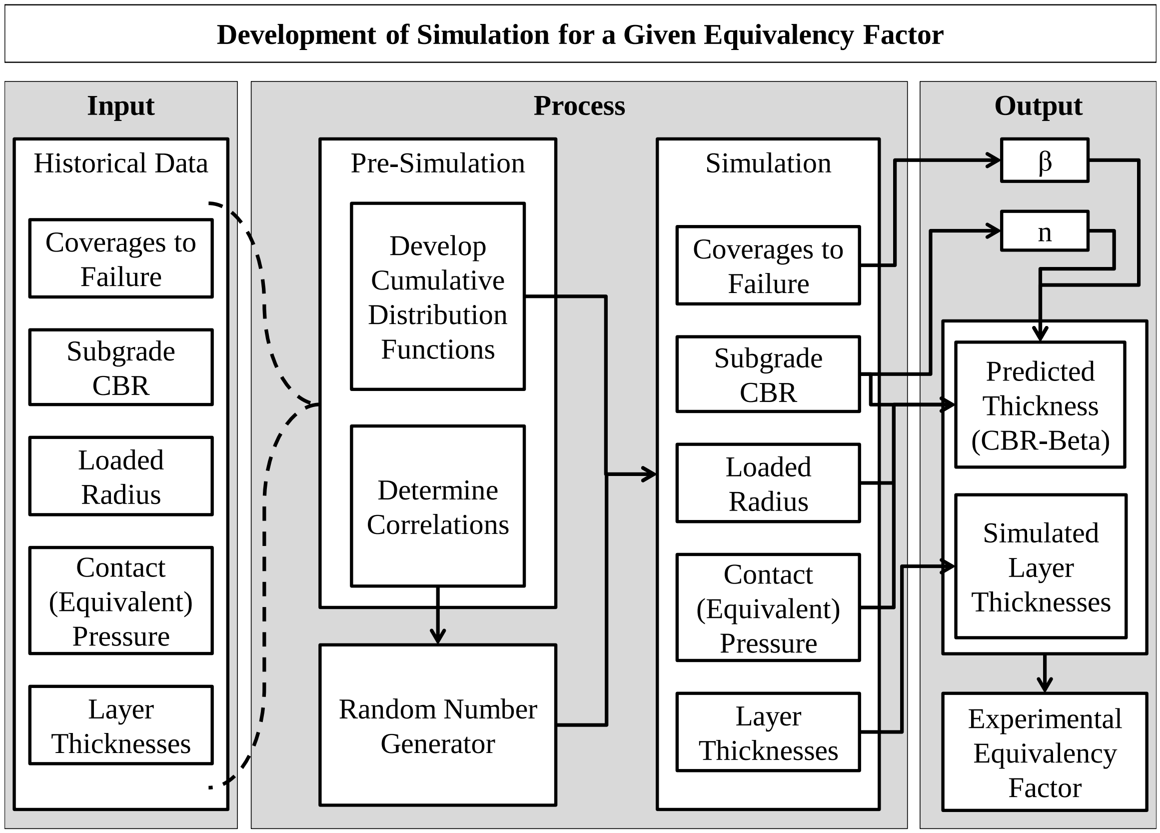

Figure 7. Overview of Statistical Simulation for Calculating Predicted and Actual Thicknesses

Since the CBR-Beta design methodology relies on the Boussinesq-Frohlich stress distribution theory to account for multiple-wheel gear assemblies, an equivalent contact pressure was reverse-calculated for each of the multiple-wheel test sections from the calculated thickness required to ensure the stresses at the top of the subgrade did not exceed the allowable stress for a given subgrade soil. The calculated thickness was determined by iterating the thickness above the subgrade until the vertical stress on the subgrade, as determined using superposition theory to account for the loads from each wheel at various evaluation points, is equivalent to the allowable stress. By using the equivalent pressure, the simulation was able to replicate representative single-and-multiple-wheel loadings.

The distributions used in the simulation were unique to the equivalency factor analyzed; each of the distributions was established using the characteristics of the actual test section corresponding to the equivalency factor in question. As a result of this methodology, several equivalency factors had fewer randomized input variables as the actual data contained no variability for a particular input variable (subgrade CBR for example). As a result, the variability seen during the simulation varied between equivalency factors depending on the number of randomized input variables. For example in Figure 8, the asphalt-stabilized GW, GP, GM, GC base course simulation randomized all of the input variables according to the distributions built from the actual data; therefore, it took approximately 400 trials on average to stabilize the cumulative average of the trials. Although some stabilized materials took more than 500 trials to stabilize, all of the materials stabilized prior to reaching 1,000 trials.

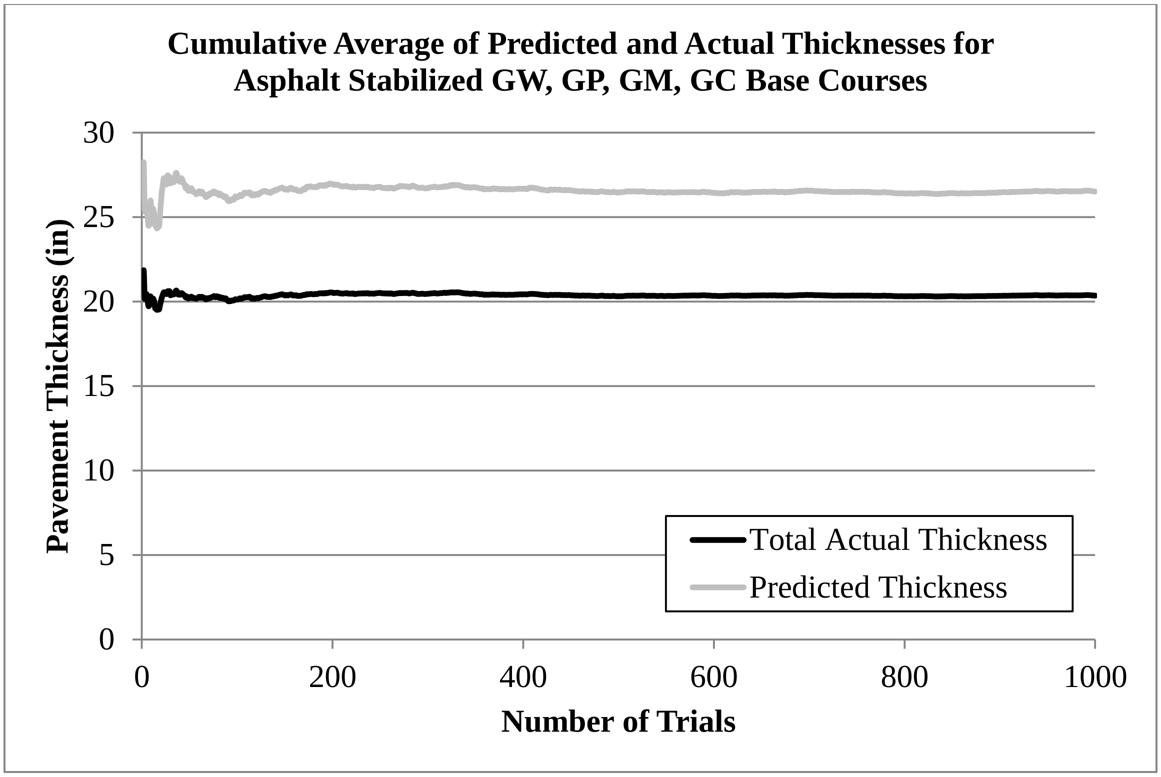

Figure 8. Cumulative Average of the Trials for the Predicted and Actual Thicknesses from the Asphalt-Stabilized GW, GP , GM, GC Base Course Simulation

Test Section Data

All of the airfield test section data incorporated into this study came from three sources: the USACE, the FAA, and Airbus. As with any meta-analysis using data from multiple testing agencies, due diligence was necessary to ensure that only data that had similar testing methodologies and failure criteria were utilized. Pavements were considered failed when rutting exceeded one inch; the coverages to failure were recorded using this distress condition. When not explicitly stated, the failure coverages were interpolated from the reports using the cross-sectional profiles and deformation curves as applicable; test sections that did not fail under trafficking were included in the analysis, but these test sections received additional scrutiny to ensure the results did not cause in extreme outliers. When the test sections are grouped to align with the equivalency factors in Table 6, each grouping provides relevant data to its respective equivalency factor; however, none of the groupings exceeded the large sample threshold.

Asphalt-stabilized

As shown in Table 7, the test section data collected for the asphalt stabilization methods aligned into two broad categories: (1) asphalt-stabilized GW, GP, GM, GC base course, and (2) asphalt-stabilized SW, SP, SM, SC subbase course. These two categories correspond to two of the five equivalency factor categories listed in Table 6 under asphalt-stabilized; no studies were found that support the other three factors. The test sections incorporated in this study involved various combinations of wheel assemblies and loads corresponding to contact pressures ranging from 105 to 278 pounds per square inch. These pressures were placed over flexible pavement structures with thicknesses above the subgrade ranging from approximately 12.6 to 39.5 inches; the subgrade CBRs ranged from 2.5 to 15.

All of the sources provided relevant data to this study; however, the Airbus source required some engineering judgment to extract acceptable data for this study. In their report, Martin et al. (2001) documented several instances where the test sections experienced immediate settlement under the pavement at the introduction of loading. With a lack of information as to the cause of this condition, eight test samples with unusually high settlement were omitted from this analysis as to avoid adversely affecting the overall results. This omission is deemed acceptable as the settling reached upwards of 1.2 inches, and it is unclear as to whether the settlement was construction or materials related. Additionally, the failure coverages from the report for the extracted test sections were interpolated from the rut versus passes graphs provided by the authors.

As an additional commentary about the data set, the four stabilized subbase test sections also contained asphalt-stabilized base courses; these tests are identified by the forward-slashed texture on Table 7 As a result of the dual stabilized layers, these tests were not included in the base course analysis; however, they were included in the subbase analysis using an equivalency factor to account for the base course stabilization. For this case, the equivalency factor for the subbase was determined from incorporating the equivalency factor calculated in this research into the equivalent thickness formulation to account for the stabilized base course.

Table 7. Full-Scale Asphalt-Stabilized Test Sections (Flexible Airfield)

		Reference	Testing Agency	Wheel Type	Load (k)	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
						Wearing	Base	Subbase			Predicted	Actual
Base Course	Asphalt Stabilized GW, GP, GM, GC	1	Airbus	Twin Tandem	146	2.4	7.9	15.8	8.0	6105	33.32	25.98
		1	Airbus	Twin Tandem	146	3.2	7.1	15.8	8.0	6105	33.32	25.99
		1	Airbus	Twin Tandem	146	4.7	5.5	15.8	8.0	6395	33.42	25.98
		1	Airbus	Twin Tandem	146	3.2	7.1	15.8	8.0	6105	33.32	25.99
		1	Airbus	Twin Tandem	146	3.2	7.1	15.8	8.0	6395	33.42	25.99
		1	Airbus	Twin Tandem	146	3.2	7.1	15.8	8.0	5814	33.21	25.99
		2	Airbus	Twin Tandem	252	3.1	9.4	0.0	15.0	2148	20.35	12.60
		2	Airbus	Triple Tandem	310	3.1	9.4	0.0	15.0	3574+	18.83	12.60
		2	Airbus	Triple Tandem	378	3.1	9.4	0.0	15.0	901	19.29	12.60
		2	Airbus	Twin Tandem	207	3.1	9.4	0.0	15.0	2487	18.66	12.60
		2	Airbus	Twin Tandem	252	3.1	9.4	7.9	10.0	3464+	25.94	20.47
		2	Airbus	Triple Tandem	310	3.1	9.4	7.9	10.0	3574+	23.59	20.47
		2	Airbus	Triple Tandem	378	3.1	9.4	7.9	10.0	3464+	26.04	20.47
		2	Airbus	Twin Tandem	207	3.1	9.4	7.9	10.0	2646+	23.37	20.47
		3	USACE	Single Wheel	75	3.1	5.8	6.0	2.5	8	33.38	14.88
		3	USACE	Single Wheel	75	3.0	7.2	13.7	5.2	90	29.95	23.88
		3	USACE	Single Wheel	75	2.8	5.9	15.0	3.0	12	32.12	23.64
		3	USACE	12-Wheel Assembly	360	3.1	5.8	6.0	2.5	425	36.57	14.88
		3	USACE	12-Wheel Assembly	360	3.0	7.2	13.7	5.2	2198	24.16	23.88
		3	USACE	12-Wheel Assembly	360	2.8	5.9	15.0	3.0	734	33.32	23.64
		4	FAA	Triple Tandem	270	5.0	4.9	29.6	4.0	16359	44.19	39.51
		4	FAA	Triple Tandem	270	5.0	4.9	29.6	4.0	15358	43.92	39.51
		4	FAA	Triple Tandem	270	5.0	4.9	8.5	8.0	4674	24.83	18.38
		4	FAA	Triple Tandem	270	5.0	4.9	8.5	8.0	8891	25.72	18.38
		4	FAA	Twin Tandem	180	5.0	4.9	29.6	4.0	13841	42.75	39.51
		4	FAA	Twin Tandem	180	5.0	4.9	29.6	4.0	11372	42.18	39.51
		4	FAA	Twin Tandem	180	5.0	4.9	8.5	8.0	10515	25.91	18.38
		4	FAA	Twin Tandem	180	5.0	4.9	8.5	8.0	7996	25.55	18.38
Subbase Course	Asphalt Stabilized SW, SP, SM, SC	3	USACE	Single Wheel	75	3.1	5.8	6.0	2.5	8	33.38	14.88
		3	USACE	Single Wheel	75	3.0	7.2	13.7	5.2	90	29.95	23.88
		3	USACE	12-Wheel Assembly	360	3.1	5.8	6.0	2.5	425	36.57	14.88
		3	USACE	12-Wheel Assembly	360	3.0	7.2	13.7	5.2	2198	24.16	23.88

References
1. High Pressure Tire Test (Airbus, 2010)
2. A380 Pavement Experimental Programme (Martin, et al., 2001)
3. Study of Behavior of Bituminous-Stabilized Layers (Burns, Ledbetter, & Grau, 1973)
4. Rutting Study of NAPTF Flexible Pavement Test Sections (Gopalakrishnan & Thompson, 2004)

According to the test reports, a majority of the FAA and Airbus test sections for the asphalt-stabilized base course material contained a crushed aggregate/gravel subbase course (Airbus, 2010; Gopalakrishnan & Thompson, 2004). For the FAA test section (identified by the dotted texture in Table 7), the subbases contained P-209, which meets the USACE's gradation requirements for base course materials. This subbase material represents an improvement over the conventional subbase material, and the U.S. Air Force and Army account for this material in its current set of equivalency factors with a factor of 2.0 (as shown in Table 6). On the other hand, the test sections from the Airbus tests contained a crushed gravel that did not meet the USACE's gradation requirements for base course materials; therefore, no equivalency factor was necessary to account for this material.

Cement-Stabilized

All of the data for the cement-stabilized factors came from the USACE testing data of which the vast majority came from a single report/experiment as shown in Table 8. This test included both channelized and distributed load patterns for the single wheel load cart. Both tests were included in this study; however, the pass-to-coverage ratios were adjusted to account for this variation. Additionally, these tests, for the most part, did not have a wearing course or a subbase course as is typically found in airfield pavements. As suggested in an internal USACE report, the test sections without a wearing course were adjusted to create an imaginary wearing course for the purpose of analysis by subtracting the minimum wearing course thickness from the predicted thickness prior to determining the base course equivalency factor (Barker, Gonzalez,

Table 8. Full-Scale Cement-Stabilized Test Sections (Flexible Airfield)

		Reference	Testing Agency	Wheel Type	Load (k)	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
						Wearing	Base	Subbase			Predicted	Actual
Base Course	Cement Stabilized GW, GP, SW, SP	1	USACE	Single Wheel	27	0.0	9.1	0.0	4.67	48	17.93	9.10
		1	USACE	Single Wheel	27	0.0	9.1	0.0	4.67	7	14.75	9.10
		1	USACE	Single Wheel	27	0.0	12.7	0.0	6.43	668	18.60	12.70
		1	USACE	Single Wheel	27	0.0	12.7	0.0	6.43	37	15.13	12.70
		1	USACE	Single Wheel	27	0.0	16.6	0.0	5.30	1000+	20.72	16.60
		1	USACE	Single Wheel	27	0.0	7.9	0.0	5.03	4	13.41	7.90
		1	USACE	Single Wheel	27	0.0	11.7	0.0	4.57	129	19.51	11.70
		1	USACE	Single Wheel	27	0.0	11.7	0.0	4.57	21	16.83	11.70
		1	USACE	Single Wheel	27	0.0	16.3	0.0	5.10	1000	21.07	16.30
		1	USACE	Single Wheel	27	0.0	8.4	0.0	5.43	18	15.31	8.40
		1	USACE	Single Wheel	27	0.0	8.4	0.0	5.43	74	17.34	8.40
		1	USACE	Single Wheel	27	0.0	11.7	0.0	5.47	120	17.91	11.70
		1	USACE	Single Wheel	27	0.0	11.7	0.0	5.47	242	18.79	11.70
		1	USACE	Single Wheel	27	1.4	11.2	0.0	5.60	636	19.72	12.60
		1	USACE	Single Wheel	27	1.0	11.2	0.0	5.60	67	16.97	12.20
		2	USACE	12-Wheel Assembly	360	2.8	21.8	0.0	3.70	10406+	35.08	24.60
		2	USACE	Single Wheel	75	2.8	21.8	0.0	3.70	200	36.75	24.60
		2	USACE	Twin Tandem	200	2.8	21.8	0.0	3.70	1810	42.89	24.60
		2	USACE	Single Wheel	75	2.8	21.8	0.0	3.70	120	35.51	24.60
		3	USACE	Twin Tandem	200	3.0	25.0	0.0	3.80	7820	47.43	28.00
		3	USACE	Twin Tandem	240	3.0	25.0	0.0	3.20	620	52.96	28.00
Subbase Course	Cement Stabilized ML, MH, CL, CH	2	USACE	12-Wheel Assembly	360	2.5	4.7	15.7	2.70	1200	37.96	22.96
		2	USACE	Twin Tandem	160	2.5	4.7	15.7	2.70	1000	42.60	22.96
		2	USACE	Single Wheel	50	2.5	4.7	15.7	2.70	120	32.75	22.96
	Cement Stabilized SC, SM	1	USACE	Single Wheel	27	1.0	3.9	11.0	5.43	1000+	20.50	15.90
		1	USACE	Single Wheel	27	1.0	3.9	11.0	5.43	217+	18.72	15.90

References

1. Performance Data for F-4 Load Cart Operations on Alternate Launch and Recovery Surfaces (Styron III, 1984)
2. Evaluation of Structural Layers in Flexible Pavements (Grau, 1973)
3. Comparative Performance of Structural Layers in Pavements, Vol II (Sale, Hutchinson, Ulery, Ladd, & Barker, 1977)

Harrison, & Bianchini, 2012). Overall, the data set contained a variety of tests that included high-pressure single wheel and lower-pressure 12-wheel assemblies.

Other Stabilization Methods

Of the additional stabilized test sections not already mentioned, only two align with categories in Table 6: lime-stabilized ML, MH, CL, CH (five samples) and Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH (two samples). These test sections are shown in Table 9. The studies involving these two materials were completed by the USACE. However, definitive conclusions would be unreasonable for the lime, cement, fly ash stabilized equivalency factor, as only two test sections are available for analysis.

As previously mentioned, crushed aggregate that meets the gradation requirements for base course materials are accounted for with an equivalency factor of 2.0 when used in the subbase under the Army and Air Force's design methodology. However, the FAA accounts for this improved material using an equivalency factor of approximately 1.4 (Federal Aviation Administration, 1995). Crushed aggregate is stronger than conventional subbase materials; therefore, it is logical to assume that an equivalency factor is necessary. As a result, this study analyzed the data to validate the current factor; however, the analysis was difficult since all of the crushed aggregate test sections contained asphalt-stabilized base courses.

Table 9. Full-Scale Miscellaneous Stabilized Test Sections (Flexible Airfield)

		Reference	Testing Agency	Wheel Type	Load (k)	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
						Wearing	Base	Subbase			Predicted	Actual
Subbase Course	Lime Stabilized ML, MH, CL, CH	1	USACE	Single Wheel	27	1.4	3.6	13.1	4.60	419	20.99	18.10
		1	USACE	Single Wheel	27	1.0	3.6	13.1	4.60	217	20.15	17.70
		2	USACE	12-Wheel Assembly	360	2.3	4.8	15.4	4.20	198	22.90	22.44
		2	USACE	Twin Tandem	160	2.3	4.8	15.4	4.20	140	25.61	22.44
		2	USACE	Single Wheel	50	2.3	4.8	15.4	4.20	40	24.61	22.44
	Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH	3	USACE	Twin Tandem	200	3.0	6.0	24.0	5.60	3660	32.38	33.00
		3	USACE	Twin Tandem	240	3.0	6.0	24.0	4.40	600	39.92	33.00
	Crushed Aggregate (P-209)	4	FAA	Triple Tandem	270	5.0	4.9	29.6	4.00	16359	44.19	39.51
		4	FAA	Triple Tandem	270	5.0	4.9	29.6	4.00	15358	43.92	39.51
		4	FAA	Triple Tandem	270	5.0	4.9	8.5	8.00	4674	24.83	18.38
		4	FAA	Triple Tandem	270	5.0	4.9	8.5	8.00	8891	25.72	18.38
		4	FAA	Twin Tandem	180	5.0	4.9	29.6	4.00	13841	42.75	39.51
		4	FAA	Twin Tandem	180	5.0	4.9	29.6	4.00	11372	42.18	39.51
		4	FAA	Twin Tandem	180	5.0	4.9	8.5	8.00	10515	25.91	18.38
		4	FAA	Twin Tandem	180	5.0	4.9	8.5	8.00	7996	25.55	18.38

References
1. Performance Data for F-4 Load Cart Operations on Alternate Launch and Recovery Surfaces (Styron III, 1984)
2. Evaluation of Structural Layers in Flexible Pavements (Grau, 1973)
3. Comparative Performance of Structural Layers in Pavements, Vol II (Sale, Hutchinson, Ulery, Ladd, & Barker, 1977)
4. Rutting Study of NAPTF Flexible Pavement Test Sections (Gopalakrishnan & Thompson, 2004)

Missing Data

As mentioned previously and shown in Table 10, several equivalency factors in use by the U.S. Air Force and Army do not have airfield test section data available to assess the accuracy of the factors. These factors are primarily in the subbase. In discussions with the AFCEC, several experts suggested that these factors without test section data were created using a combination of highway data and expert opinion (Personal Communication, 15 Aug 2013). Therefore, the current study did not address these factors, as alternative sources of quantitative data are necessary.

Table 10. Summary of Test Section Counts and Missing Equivalency Factor Data

Course	Stabilizer	Test Section Count
Base	All-Bituminous Concrete	Not Evaluated
	Asphalt-stabilized GW, GP, GM, GC	28 (16)
	Cement-stabilized GW, GP, SW, SP	21
	Cement-stabilized GC, GM	0
Subbase	All-Bituminous Concrete	0
	Asphalt-stabilized GW, GP, GM, GC	0
	Asphalt-stabilized SW, SP, SM, SC	4
	Cement-stabilized GW, GP, SW, SP	0
	Cement-stabilized SC, SM	2
	Cement-stabilized ML, MH, CL, CH	3
	Lime Stabilized ML, MH, CL, CH	5
	Lime Stabilized SC, SM, GC, GM	0
	Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH	2
	Lime, Cement, Fly Ash Stabilized SC, SM, GC, GM	0
	Lime, Cement, Fly Ash Stabilized Unbound Crushed Stone	0
	Lime, Cement, Fly Ash Stabilized Unbound Aggregate	0
	Crushed Aggregate	8

General Results

As previously mentioned, due to limited data availability, the current study investigated three base course and six subbase course equivalency factors. For the sake of brevity, the results from the study are summarized in a later section. However, to provide clarity to the reader, an example of the intermediate results and the calculations compiled for each equivalency factor are presented in the subsequent section.

Results Developed for Each Equivalency Factor

The study for the cement-stabilized GW, GP, SW, SP base course equivalency factor considered 21 test sections, all from the USACE. All of the actual data points, as shown in Figure 9, appeared to indicate that the median equivalency factor value was slightly above 1.10. Although the minimum and maximum values for the simulation are slightly lower and higher than the actual data, respectively, the cumulative distributions appear to track together. The mean and median calculated for the actual data were 1.19 and 1.12, respectively. In comparison, the mean and median were 1.19 and 1.20, respectively, for the simulated data. The minor disparities between the actual and simulated values are within the standard error of the sample.

The two distributions were further analyzed using the Wilcoxon Rank-Sum Test for non-parametric analyses. The null hypothesis for this test was that no statistical difference existed between the two samples; conversely, the alternative hypothesis assumed that there was a statistical difference between the samples. Using a two-tail test and a calculated z-score of 0.325, it was determined that the null hypothesis cannot be rejected at the 95% confidence level. Given this conclusion, it was verified that the simulation was an accurate representation of the actual data; therefore, the simulated data could be used to determine the equivalency factor for cement-stabilized GW, GP, SW, SP base course material.

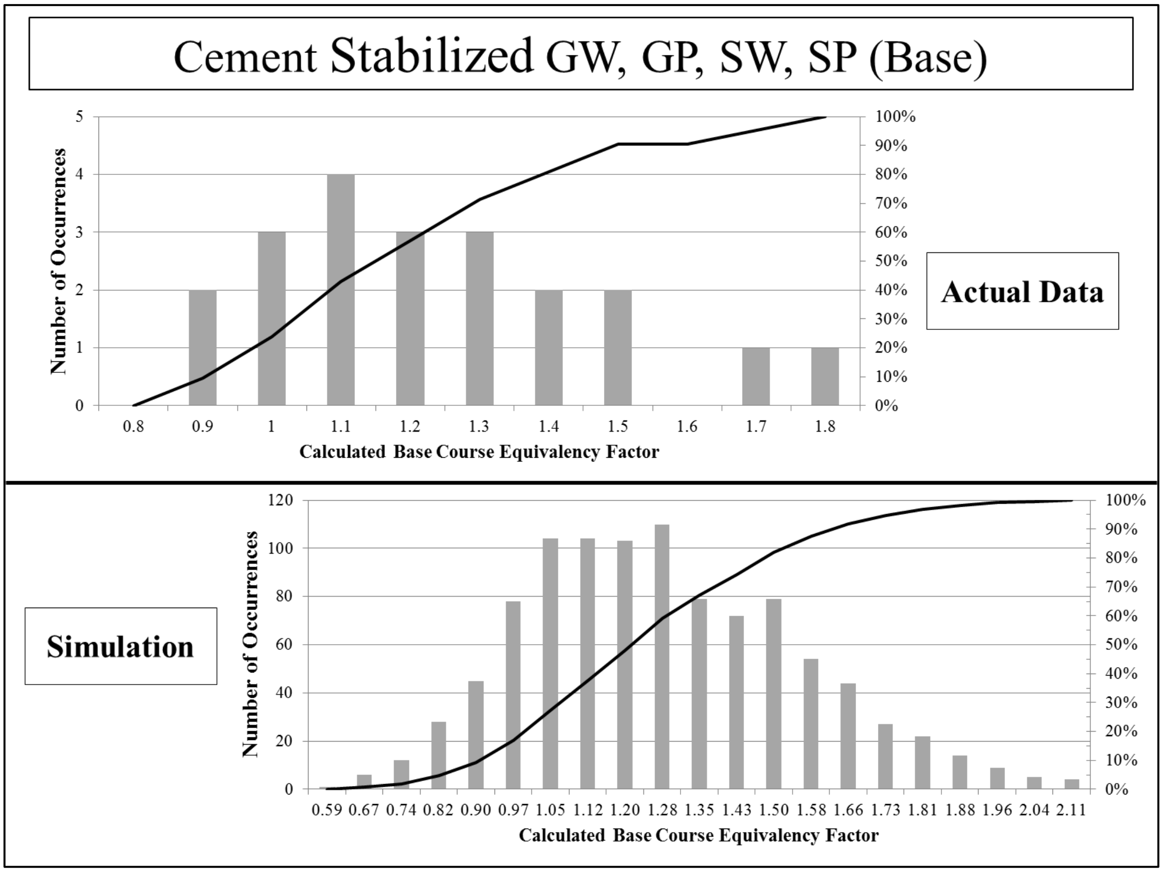

Figure 9. Comparison Between Actual and Simulated Data for Cement-Stabilized GW, GP, SW, SP (Base Course Equivalency Factor)

As another example of correlation between the distributions, the cumulative distribution from both sets of data are overlaid on the same plot in Figure 10. Due to the different sample sizes and bin locations on the histograms, the curves do not match perfectly. However, the curves tend to follow the same pattern when the cumulative distribution is between 10 and 95 percent. The visual overlay serves as further support to the statistical analysis mentioned in the previous paragraph.

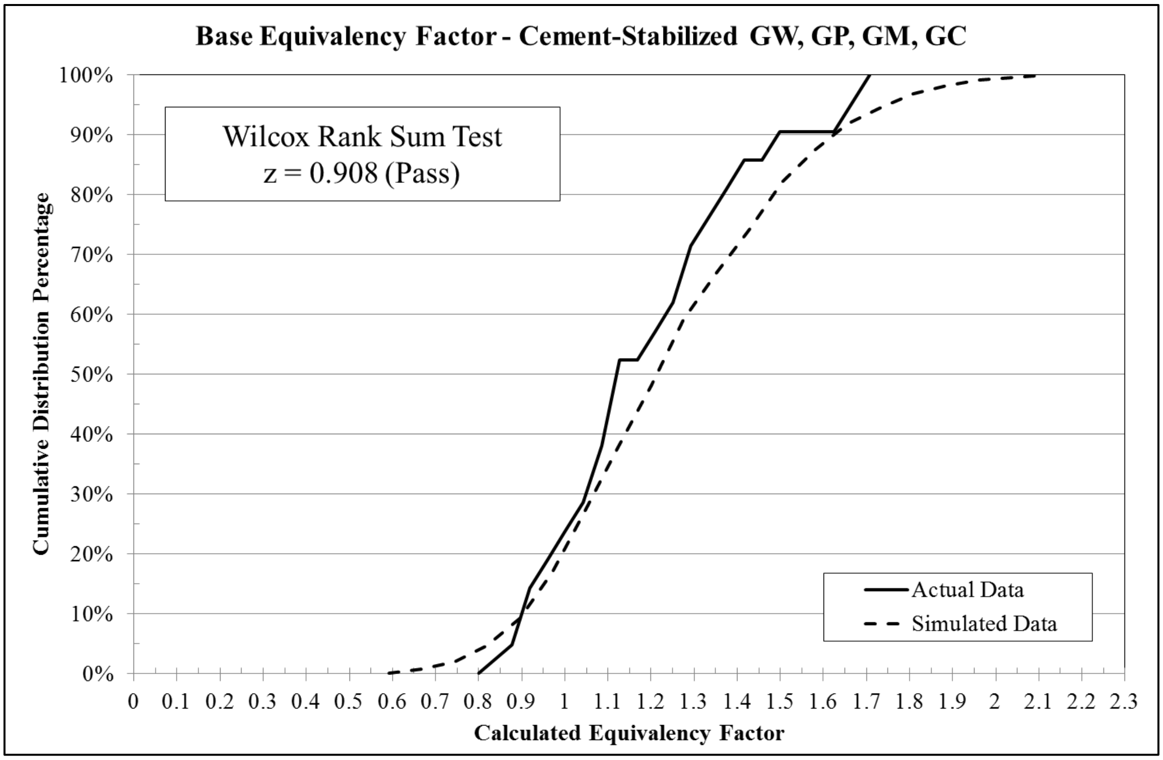

Figure 10. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Cement-stabilized GW, GP, SW, SP (Base Course Equivalency Factor)

The simulated data was decomposed into five percent increments from zero to one using the calculated equivalency factors for the 1,000 trials. With the percentile breaks, the final equivalency factor for the cement-stabilized base course was determined by optimizing the factor in terms of its ability to maximize the predictability of the overall CBR-Beta design model. Throughout the course of the literature review for this overall research effort, 157 test sections were compiled to evaluate the CBR-Beta design model; of these test sections, roughly 48 percent of the data represented stabilized pavements. By using the percentile equivalency factors to adjust the equivalent thickness for the 21 cement-stabilized base courses, the effect each percentile had on the predictability of the overall CBR-Beta model could be evaluated; for ease of analysis, the model was optimized in terms of R^2. As shown in Figure 11, the optimal equivalency factor, which produces the highest R^2 value, occurred around 0.94. Since this value is less than 1.00, it was rounded to the minimum equivalency factor as recommended by Sale et al. (1977). By rounding the equivalency factor to 1.00, a reduced R^2 value and an additional amount of uncertainty are implicitly accepted. The difference in additional uncertainty, as shown in Figure 10, between 0.94 and 1.00 is approximately 9 percent; the uncertainty for the 1.0 value is approximately 24 percent.

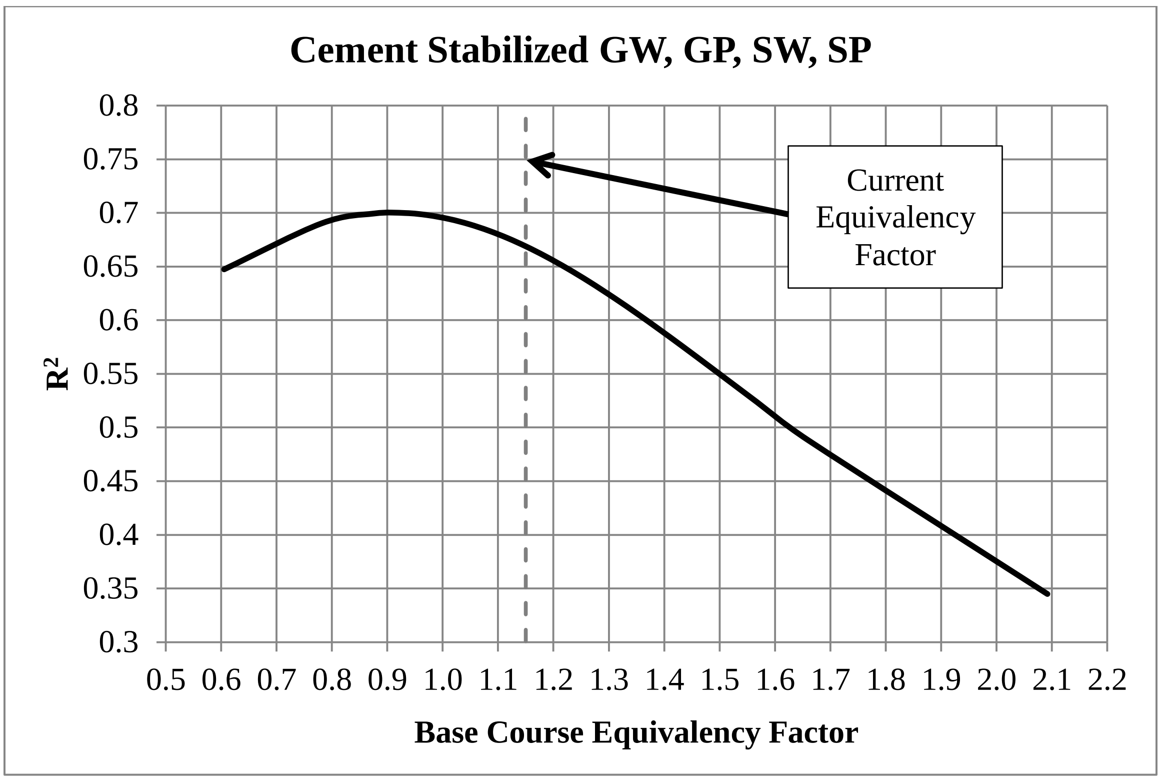

Figure 11. Optimization Output for Cement-Stabilized GW, GP, SW, SP (Base Course)

By reducing the equivalency factor from 1.15 [which is the current value specified in UFC 3-260-02, as shown by dashed-line in Figure 11] to 1.0, the predictability of the model can be increased 5.3 percent. An equivalency factor of 1.0 should not imply that the cement-stabilized base courses offer no improvement over conventional base courses; rather, it should stress the relative necessity of calibrating a set of equivalency factors to the design and evaluation method for which it will be used. For example, the USACE equivalent thickness procedure credits stabilized base courses as if it possessed a 100 CBR rather than the 80 CBR of a conventional base course. This distinction results in a reduction in layer thickness relative to a conventional base course; however, it offers no additional reduction relative to a high-quality base course material.

Aggregated Results

The intermediate results and calculations for each of the other equivalency factors are not included in this article; however, each of the remaining equivalency factors was analyzed using the same methodology from the previous section. Throughout the study, each of the simulations produced cumulative distributions that tracked with the actual data. This statement is further reinforced by the fact that each simulation failed to reject the null hypothesis for the Wilcoxon Rank-Sum Test, thereby validating the use of the simulation to model the actual data.

Consistency in methodology was maintained throughout the study; however, a slight variation was necessary for the doubly-stabilized test sections. These test sections were comprised of an asphalt-stabilized base course and either an asphalt-stabilized or crushed aggregate subbase course. These doubly-stabilized test sections were imperative to analyze as these test sections represented the entirety of the data for each of the respective subbase materials. To analyze each of these stabilized subbases, an equivalency factor of 1.25 was used to convert the asphalt-stabilized base course to an equivalent thickness. As shown in Table 11, this value corresponds to the 80 percent confidence equivalency factor for the actual and simulated data sets for this stabilized base course.

Table 11. Aggregated Results for Actual and Simulated Data by Equivalency Factor

		Actual Test Section Data					Simulation Data					Wilcoxon-Rank Sum Test (Z-Core)	Optimized Equivalency Factor
		n	Min	Mean	Median	Max	n	Min	Mean	Median	Max		
Base	Asphalt-stabilized GW, GP, GM, GC	16	0.97	1.38	137	1.83	1000	0.65	1.19	1.20	1.95	0.724	1.23
Base	Cement-stabilized GW, GP, SW, SP	21	0.88	1.12	1.19	1.71	1000	0.99	1.59	1.62	2.15	0.325	0.94
Subbase	Asphalt-stabilized SW, SP, SM, SC	4	0.72	2.47	2.44	4.27	1000	0.73	2.45	2.16	4.62	0.283	1.55
Subbase	Cement-stabilized ML, MH, CL, CH	3	1.62	1.94	1.95	2.25	1000	1.66	2.00	2.00	2.26	0.222	2.20
Subbase	Cement-stabilized SC, SM	2	1.26	1.34	1.34	1.42	1000	1.26	1.36	1.37	1.42	0.000	1.26*
Subbase	Lime Stabilized ML, MH, CL, CH	5	0.91	1.17	1.14	1.57	1000	0.48	1.21	1.20	1.92	0.500	1.12
Subbase	Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH	2	0.97	1.13	1.13	1.29	1000	1.00	1.15	1.16	1.26	0.000	1.26*
Subbase	Crushed Aggregate (P-209)	8	1.02	1.48	1.41	2.12	1000	1.07	1.20	1.14	1.65	0.980	1.33

*Sample did not converge on an optimal solution; maximum of minimum value reported

Analysis of Results

As shown in Table 12, the optimized equivalency factors determined from this study were rounded to the nearest five-hundredth increment, thus forming the recommended equivalency factors from this study for the CBR-Beta method. Using these recommended values, a reverse-calculated percent confidence was determined based upon the corresponding equivalency factor’s percentiles of the simulated data. This percent confidence reflected the percent of the simulated trials that resulted in equivalency factors higher than the recommended value. With the assumption that the simulation represented the actual population distribution for the stabilized layer, the percent confidence would reflect the percentage of occurrences in which the use of the factor in real-world applications would result in higher actual equivalencies. For pavements with higher actual equivalencies, the recommended factor would prove conservative.

Table 12. List of Evaluated Equivalency for Flexible Airfield Pavements

			Equivalency Factors					
Course	**Stabilizer**	**Count**	**Army/AF**	**USMC/USN**	**FAA**	**ICAO**	**Recommended**	**% Confidence**
Base	Asphalt-stabilized GW, GP, GM, GC	28	1.00	1.50	1.60	1.50	1.25	82%
	Cement-stabilized GW, GP, SW, SP	21	1.15	1.50	1.20	1.50	1.00	76%
Subbase	Asphalt-stabilized SW, SP, SM, SC	4	1.50	1.00	2.30	1.00	1.55	65%
	Cement-stabilized ML, MH, CL, CH	3	1.70	1.20	1.60	1.00	2.20	5%
	Cement-stabilized SC, SM	2	1.50	1.20	1.60	1.00	1.25	100%
	Lime Stabilized ML, MH, CL, CH	5	1.00	1.20	1.00	1.00	1.10	65%
	Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH	2	1.30	1.00	1.00	1.00	1.30	0%
	Crushed Aggregate	8	1.00	1.00	1.40	1.00	1.35	14%

As alluded to previously, each design method relies on different formulations for equivalent thickness and makes different assumptions in its respective design process. As such, a direct comparison of the factors would be inappropriate; however, the factors can be analyzed looking at the trends within each set. For example, the base course equivalency factors for this study ranked the asphalt-stabilized material higher than the cement-stabilized material. With the exception of the FAA, the other three sets of factors suggested the asphalt-stabilized base course was not significantly stronger than the cement-stabilized base course. Furthermore, the FAA suggests the improvement with asphalt-stabilized base is approximately 33 percent more than with cement-stabilized; the recommended factors for this study suggested this increase slightly lower at 25 percent. For the base courses, the recommended factors from this study tend to agree with the factors from the FAA. As a general note, the FAA specifies its equivalency factors as a range of values as a function of the modulus value; the values presented in Table 12 reflect a mean value.

Due to the limited sample sizes of the subbase materials, the confidence percentages for the recommended equivalency factors result in significantly reduced values relative to the confidence of the base course factors. In conversations with the AFCEC, the subject matter experts stated that the Air Force does not typically use subbase stabilization; therefore, lower confidence rates were considered more acceptable in the subbase than in the base course (Personal Communication, 9 Dec 13). Inevitably, further investigation is necessary to increase the sample size and, as a result, increase the confidence of the estimates.

Summary and Conclusions

When compared to the current equivalency factors used by the U.S. Army and Air Force, the results of this study represent a significant refinement to the equivalency factors used for flexible airfield pavement design and evaluation. For the 157 test sections in the database, this refinement resulted in a seven percent increase in R^2 for the overall CBR-Beta design method. Additionally, by incorporating the recommended factors from this study, the median absolute percentage error was reduced by 14.6 percent. By interpreting these two statistical measures, these recommended factors will lead to more accurate pavement evaluations and designs. To U.S. Army and Air Force, these revised equivalency factors result in a less conservative set of equivalency factors, which can lead to thinner pavements and thus reduced initial construction costs.

This refinement comes at a good time as the CBR-Beta design methodology is being implemented as the standard for the U.S. Air Force and Army flexible pavement program; it was already incorporated into the latest version (2.09.02) of the USACE's Pavement-Transporation Computer Assisted Structural Engineering (PCASE) software. These factors are calibrated and intended for use with this design methodology; however, adaptations can be made to apply these factors to other design methodologies. Although equivalency factors are overly simplistic to describe the structural benefits of stabilized soils, these factors are imperative for design and evaluation in contingency environments and for the evaluation of pavements with substandard soils.

Disclaimer

The views expressed in this article are those of the author and do not reflect the official policy or position of the United States Air Force, Department of Defense, or the United States government.

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IV. Journal Article: Life-Cycle Cost Comparison of Various Flexible Airfield Pavement Designs Methodologies

The journal article presented in this chapter was submitted for publication to *The Military Engineer (TME)*. The journal article presents the results of the life-cycle cost comparison of the various flexible airfield pavement design methodologies from the Department of Defense (DoD) and the Federal Aviation Administration (FAA). While the content of this chapter is the same as the journal submission, formatting adaptations have occurred for inclusion in this thesis. Further support and information regarding the content contained in this article is available in Appendices B, C, and D.

Life-Cycle Cost Comparison of Various Flexible Airfield Pavement Designs Methodologies

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Abstract

With over 1.5 billion square feet of airfield pavements in its portfolio, the U.S. Air Force has a vested financial interest in refining its design, maintenance, and inspection criteria to increase the efficiency of its infrastructure investment. As part of its strategic pavement assessment, the Air Force is currently moving to adopt the new design method (CBR-Beta) developed by the U.S. Army Corps of Engineers on the basis that the new methodology more accurately represents the performance of historical experimental data and produces thinner flexible pavements. This conclusion is based upon the assumption that thinner pavements equate to cheaper pavements; however, this assertion fails to account for the idea that thinner pavements support fewer aircraft passes. The focus of this research effort was to incorporate the service life component into the analysis and compare the new design model with the status quo, as well as the models in use by the Federal Aviation Administration, in terms of life-cycle cost.

Key Words (Subject Headings)

Flexible Pavement; Pavement Design; Life-Cycle Cost Comparison; Stabilized Soils

Headline

Engineers leverage life-cycle cost analysis philosophy to implementing pavement design methods.

Introduction

The Department of Defense (DoD) is the largest owner and operator of airfield pavements in the United States; the Air Force alone has over 1.5 billion square feet of airfield pavements in its portfolio (Air Force Civil Engineer Center, 2012). Based on the size of the portfolio, the DoD continuously invests hundreds of millions of dollars each fiscal year to ensure its pavements continue to support the flying mission. Although these investments include routine maintenance actions, such as rubber removal and joint sealants, annual investments often include more extensive full-depth repairs when the pavement reaches the end of its service life. Since full-depth repairs, as well as new construction, require a large expenditure of funding to complete, the DoD has a vested interest in utilizing pavement design methods that specify pavement thicknesses sufficient to support aircraft operations throughout the design life while minimizing cost. This research effort focused on addressing these two issues for flexible airfield pavements as part of the Army and Air Force's strategic review of their pavements program.

Pavement Design Methods

In the early 1940s, the U.S. Army Corps of Engineers (USACE) assumed responsibility for design and construction of all military airfields (Ahlvin, 1991). As

World War II started and the necessity for heavy bombers became evident, USACE realized a pavement design method was necessary to support heavier loads. It examined several promising methods for pavement design before ultimately approving the California Bearing Ratio (CBR) design methodology that was being used by state highway officials. USACE conducted full-scale, accelerated pavement testing in the 1940s to adapt the empirical CBR design method for airfield use (Gonzalez, Barker, & Bianchini, 2012). Later tests incorporated experiments with larger aircraft, multiple wheel configurations, mixed traffic, different coverage levels, and stabilized soils (Ahlvin, 1991). Based on the full-scale testing in the 1970s, USACE aggregated the empirical data to formulate the CBR-Alpha method for designing pavements. This method remained largely unchanged until recently, as USACE is finalizing an update (CBR-Beta) to the current CBR equation based on the Boussinesq-Frohlich stress distribution and is planning to publish it in the next update to Unified Facilities Criteria (UFC) 3-260-02.

In 1975, USACE developed its layered-elastic design method based upon Burmeister's layered theory (Ahlvin, 1991). Although USACE presented this design methodology as an optional method for flexible pavements, UFC guidance recommends its use as the primary method for stateside locations. Building upon USACE's work, the Federal Aviation Administration (FAA) developed its own layered-elastic design (LED) method in the mid-1990s known as LEDFAA (Brill, 2012a). The FAA further refined its layered-elastic design method with the release of FAA Rigid and Flexible Iterative Elastic Layered Design (FAARFIELD) in 2009. With the release of FAARFIELD, the FAA stopped using the CBR method for design (Federal Aviation Administration, 2009).

Research Method

To compare the design methods, scenarios were developed based on varying the subgrade CBR, aircraft passes, and base course material. As an example of this variation, a design aircraft was considered with 10,000 passes on a flexible pavement comprised of conventional base and subbase courses over a subgrade with a CBR of 3 percent. The C-17, F-15E, and the 777 were used as the design aircraft. The pavements for the 81 resulting scenarios were designed using the seven different design methods: (1) CBR-Alpha with current equivalency factors, (2) CBR-Beta with current equivalency factors, (3) CBR-Alpha with modified equivalency factors, (4) CBR-Beta with modified equivalency factors, (5) FAARFIELD, (6) LEDFAA using the FAA's previous criteria from AC 150/5320-6D, and (7) USACE's LED. The modified equivalency factors for CBR-Alpha and CBR-Beta were derived from an earlier portion of the effort.

Utilizing the predicted pavement thickness data from the seven design methods, the difference in thicknesses and cost for each of the methods relative to CBR-Alpha with current equivalency factors (i.e., status quo) were compared. The passes to failure were then calculated for each of the predicted thicknesses using the status quo design method. This step accounts for the fact that 10,000 passes corresponds to significantly different pavement thicknesses for each of the methods; therefore, each of the methods can be analyzed in terms of service life (passes to failure) and usage costs (cost per pass). As a general note, this analysis only considered aircraft loadings as sole source of deterioration (i.e., climate and maintenance actions were not considered). This assumption was taken to simplify the analysis, and on the basis that if the pavements designed using the various methods were all subjected to the same environmental effects, loadings, and maintenance efforts, then the only distinguishing difference between the pavements would be thickness above the subgrade. As such, the thickness would be the only difference in determining the service life of the pavement.

General Results

The predicted thicknesses and initial construction costs for each model were compared to the status quo by utilizing a cumulative percentage difference. Based on the analysis, the USACE's LED was the only design method that produced thicker pavements than the status quo over the 81 scenarios; LEDFAA and FAARFIELD both produced similar results with the results being slightly less than the status quo. All variations of the CBR-Beta method, as well as the modified CBR-Alpha method, produced significantly thinner pavements than the status quo.

Similar results from the thickness differences were seen when the models were compared in terms of initial construction cost as shown in Figure 12; however, the results for FAARFIELD were significantly lower in terms of percentage difference relative to differences in predicted thickness. This method produced thinner pavements for all 81 scenarios; however, for a large portion of the samples, the FAA model produced thicker pavements than the status quo. The initial assumption based on this observation was that this model would produce similarly priced pavements relative to the status quo, but this assumption proved inaccurate when the cost data from RSMeans, a construction industry cost handbook, were incorporated into the analysis. This is because FAARFIELD produced thicker subbase courses and thinner base courses compared to the status quo; the FAA adopted new minimum thickness criteria for the wearing course and base course with the introduction of FAARFIELD. Since subbase material is less expensive than base course material, the FAA models with thicknesses similar to the status quo result in less initial construction costs.

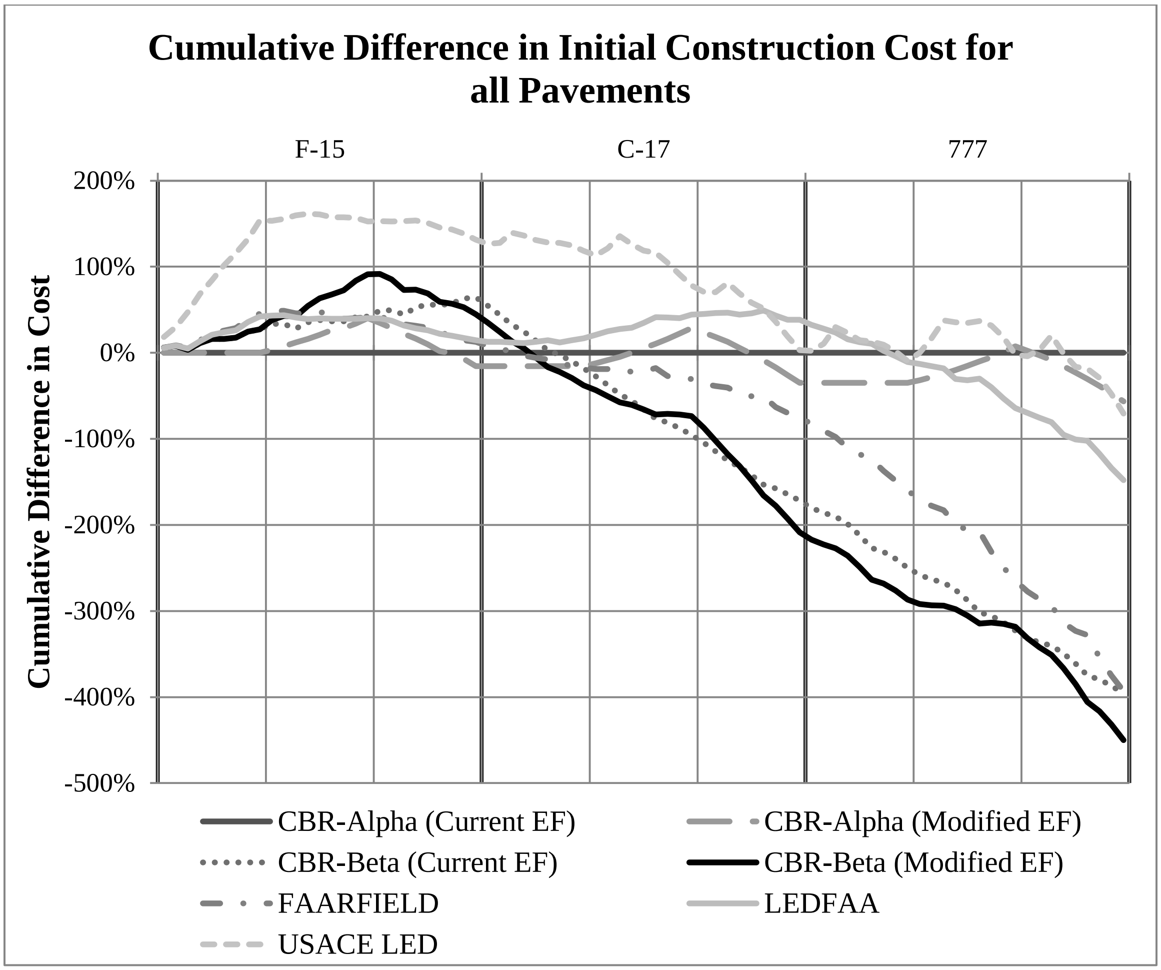

Figure 12. Cumulative Difference in Initial Construction Cost for all Pavements (n= 81)

Analysis of Results

The design methods were standardized by back-calculating the service life (passes to failure) using the CBR-Alpha method in terms of the respective predicted thickness for each of the design methods for each scenario. Based on this analysis, it was shown that LEDFAA, FAARFIELD, and USACE's LED pavements had a service life that greatly exceeded the status quo over the 81 scenarios, whereas both CBR-Beta models and the modified CBR-Alpha model resulted in lower service lives. It is worth mentioning that the LEDFAA and FAARFIELD models appeared to produce longer service lives; however, when analyzed on a smaller scale, it was apparent that the models had localized regions where the predicted service life varied significantly from the status quo at a rate demonstrably different from the other scenarios within the two models.

Combining the service life and construction cost data together, the differences between the design methods in terms of operating cost (i.e., cost per pass) were determined. This metric allows for a standardized comparison among the design methods without relying solely on initial costs. As shown in Figure 13, the status quo produced the least expensive pavements over the service life of the pavement. This conclusion stems from the fact that the CBR-Alpha model is inherently more conservative than the other models and the differences in initial construction costs are relatively insignificant to warrant a more representative pavement design method. This statement is further supported by an earlier portion of the research that compared the relative accuracy of the different models in predicting the passes to failure for the given models; this work demonstrated that the more accurate the model, the more costly per pass the pavement was during the analysis.

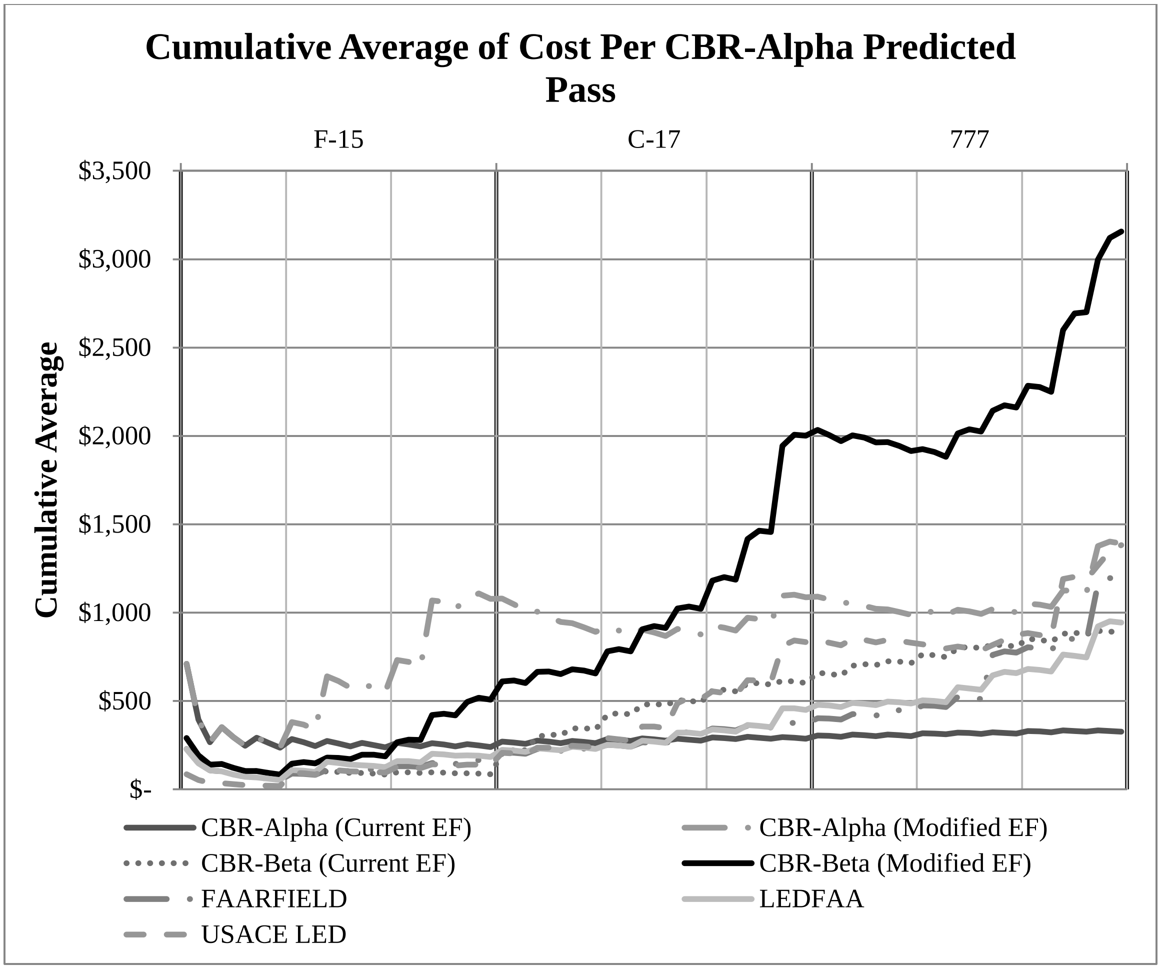

Figure 13. Cumulative Average of Cost Per CBR-Alpha Predicted Pass (n = 81)

Summary and Conclusions

The results of this analysis shift the thinking with regard to the formulation of pavement design methods. By selecting a design method based solely on initial cost, predicted thickness, or accuracy in relation to failure passes of experimental data, decision-makers often overlook the long-term implications of such a decision. For example, the modified CBR-Beta model produced the thinnest pavements at the lowest cost and appears to offer the highest predictability with regard to historical experimental data; however, when the life-cycle of the pavement is considered, the modified CBR-Beta method results in the shortest service life of all the methods. Therefore, these pavements would require additional full-depth repairs more frequently. This assertion holds true even beyond the scope of this particular research effort, as the overarching push to embrace asset management principles requires decision-makers to look beyond initial costs and analyze the life-cycle costs of their infrastructure in an effort to maximize the efficiency of their investments.

Disclaimer

The views expressed in this article are those of the authors and do not reflect the official policy or position of the United States Air Force, Department of Defense, or the United States government.

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V. Journal Article: Effect of Layer Thickness and Subgrade Depth on the Concentration Factor for Flexible Airfield Pavements using the CBR-Beta Design Method

The journal article presented in this chapter is intended for submission to the *Journal of Transportation Engineering (American Society of Civil Engineers)*. The journal article presents the findings from the analysis of the formulation of the concentration factor for use in design pavements and evaluating the vertical stress on the subgrade. While the content of this chapter is the same as the journal submission, formatting adaptations have occurred for inclusion in this thesis. Further support and information regarding the content contained in this article is available in Appendix E, F, and G.

Effect of Layer Thickness and Subgrade Depth on the Concentration Factor for Flexible Airfield Pavements Using the CBR-Beta Design Method

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Abstract

In the late 2000s, the U.S. Army Corps of Engineers (USACE) shifted its flexible airfield design methodology from the primarily empirical California Bearing Ratio (CBR) Alpha method to the mechanistic-empirical CBR-Beta method. This update in the design method came with additional challenges not seen in with the CBR-Alpha method; these challenges include the formulation of Frohlich's concentration factor and the assumption of homogeneity throughout the multi-layer pavement structure. This research sought to address both of these challenges by expanding on the work of Bianchini (2014) using more in-depth iterative analysis with a larger sample size. Ultimately, this research was able to demonstrate that the use of Frohlich's concentration factor with CBR-Beta was more representative as a two-layer model than as a homogenous, single-layer model. Additionally, this research demonstrated that the β factor should be stress-derived as opposed to failure-derived.

Key Words (Subject Headings)

Equivalency Factors; Flexible Pavements; Military Airfields; Pavement Design; Stabilized Soils; CBR-Beta; Concentration Factor

Background

This article investigated formulation of Frohlich's concentration factor for use with the U.S. Army Corps of Engineers' (USACE) California Bearing Ratio (CBR) Beta flexible airfield pavement design method. A more detailed summary of the objectives are presented later, but to introduce the topic in more detail a brief summary of the evolution of the USACE's CBR method is included. Additionally, this introduction includes a summary of the derivation of the CBR-Beta method.

Early on in the development of airfield design criteria, the U.S. Army Corps of Engineers (USACE) was at the forefront. This leading role grew out of the necessity to design pavements for the rapidly expanding bomber fleet of the U.S. Army Air Corps during World War II. After the war ended, the USACE continued to advance airfield pavement design through extensive full-scale, flexible pavement testing; this testing lasted until the 1970s with the development of the California Bearing Ratio-Alpha (CBR-Alpha) design method. Since then, the USACE has significantly scaled back its testing programs and continues to utilized the CBR-Alpha design method with little change (Ahlvin, 1991). With the USACE testing program scaled back, other agencies, such as the Federal Aviation Administration (FAA), established its own testing programs.

The FAA began experimenting with full-scale test sections in the early 2000s at the National Airport Pavement Testing Facility (NAPTF) in an effort to provide reliable performance data on newer, heavier aircraft, such as the Boeing 777 and the Airbus 380. Both aircraft were produced after USACE's Multi-Wheel Heavy Gear Load (MWHGL) Test, thus no full-scale data existed for these aircraft (Brill, 2012b). Using the testing data, the FAA evaluated and calibrated its layered-elastic analysis program to accommodate heavier aircraft with complex gear configurations; this same testing exposed holes in the CBR-Alpha method (Mallick & El-Korchi, 2013). Seeing sufficient evidence to abandon the CBR-Alpha design method, the FAA fully adopted layered-elastic design (FAARFIELD) as its primary design method in 2009 with the publication of A/C 150/5320-6E (Federal Aviation Administration, 2009).

The FAA's abandonment of the CBR-Alpha design method was a calculated decision. FAAFIELD, or layered-elastic design in general, requires standard materials with known material properties. At airports across the U.S., the ability to acquire standard materials is relatively easy. However, in contingency environments where the U.S. military operates, standard materials are often harder to find. Given the conditions and time constraints under which the military operates, the CBR method is preferred as it is less complex and can accommodate substandard materials; the ability to accommodate substandard materials is noted as the primary reason for selecting CBR methods over a layered-elastic method for evaluation (Air Force Civil Engineer Center [AFCEC], Personal Communication, 22 Apr 2013).

In an effort to revise the CBR criteria to accommodate heavier aircraft, USACE undertook a research project in the mid-to-late 2000s at the request and with significant funding the Air Force Civil Engineer Center (AFCEC); this project eventually led to the development of the CBR-Beta design method. The catalyst for the change came in a 2004 report that concluded the USACE's CBR-Alpha method "cannot adequately compute or predict pavement damage caused by new large aircraft" (Information and Technology Platform for Transport, Infrastructure, and Public Space (CROW), 2004, p. 17). After scrutinizing the criticism, the USACE realized the primary issue with CBR-

Alpha was the alpha-factor, as the research team felt it did not adequately represent multi-wheel aircraft. Additionally, the USACE felt the CBR-Alpha method was too empirical. To remedy these issues, the USACE came to the conclusion that the only solution was to reformulate the CBR design procedure (Gonzalez, Barker, & Bianchini, 2012).

Wanting to instill a mechanistic basis for the design method, Gonzalez et al. (2012) began the reformulation with the Boussinesq-Frohlich equation for vertical stress caused by a point load. The rationale for starting with this equation was to change the multi-wheel criteria from a subgrade deflection-based model to a stress-based model; the equivalent single-wheel load (ESWL) concept in the CBR-Alpha method was based upon deflection criteria. As a first step, they combined the Boussinesq-Frohlich stress model (directly under a point load) shown as Equation 8 with the initial CBR design equation from the 1940s shown as Equation 9 (Gonzalez, Barker, & Bianchini, 2012):

$$\sigma_t = \sigma_o \left[1 - \frac{1}{\left(\sqrt{1 + \left(\frac{r}{t} \right)^2} \right)^n} \right] \tag{8}$$

where,

σ_t = Vertical Stress of Point of Interest
σ_o = Applied Stress on the Loaded Area
t = Depth of the Point of Interest
r = Radius of the Loaded Area
n = Frohlich's Concentration Factor

$$t = k\sqrt{P} = k\sqrt{p\pi r^2} \tag{9}$$

where,

t = Thickness (Above Subgrade)
k = Constant Derived as a Function of Subgrade CBR and Tire Contact Pressure
P = Applied Load
p = Applied Contract Pressure
r = Radius of the Loaded Area

During the formulation of the new model, Gonzalez et al. (2012) created a new variable, β (beta), to link the eventual design model to allowable subgrade vertical stress. The new variable was then substituted into the design equation, and the design equation was then solved for in terms of β. By solving for β, they were able to utilize historical test section data to calibrate β in terms of vertical stress and applied coverages. With β now a function of both vertical stress and coverages, the previously used α (alpha) factor was no longer necessary. The incorporation of β into the derivation of the new design equation ultimately led to the CBR-Beta design equation shown as Equation 10:

$$\frac{t}{r} = \frac{1}{\sqrt{\left(\frac{1}{1-\frac{\beta CBR}{\pi\rho}}\right)^{\frac{2}{n}} - 1}} \tag{10}$$

where,

t = Thickness of Above Subgrade
r = Contact Radius
β = Beta Factor (Function of Coverages and Stress)
CBR = California Bearing Ratio
ρ = Single-Wheel Contact Pressure (Equivalent Pressure for Multiple-Wheel Gear Assemblies)
n = Stress Concentration Factor

As a general note, this design equation is applicable for single-wheel application. Multiple-wheel gear assemblies require an iterative process involving the evaluation of multiple points under the assembly using superposition to account for the vertical stresses on the subgrade (or layer of interest) from each wheel; the depth of the subgrade is adjusted until the calculated vertical stress equals the allowable stress (shown as Equation 11):

$$\sigma_{allowable} = \frac{\beta CBR}{\pi} \tag{11}$$

Frohlich's concentration factor was developed as a method to account for differences in soil properties that the original Boussinesq equation did not consider. The Boussinesq equation is based upon a concentration factor of three; however, by varying the concentration factor according to soil, engineers were able to more closely predict the measured stresses (Olmstead & Fischer, 2009). Typically, the concentration factor is determined as a point estimate for a given type of soil. This assumption proves problematic for flexible pavements that are typically comprised of multiple layers of different soil properties; improperly assigning the concentration factor can result in misleading stress values, and ultimately early pavement failures in the case of an under predicted concentration factor; this can be seen in Figure 14. As such, the USACE utilized test section data from the Stockton Field Tests to model the concentration factor in terms of subgrade (or subbase) CBR as shown in Equation 12 (Gonzalez, Barker, & Bianchini, 2012; Bianchini, 2014):

$$n = 2\left(\frac{CBR}{6}\right)^{0.1912} \quad (12)$$

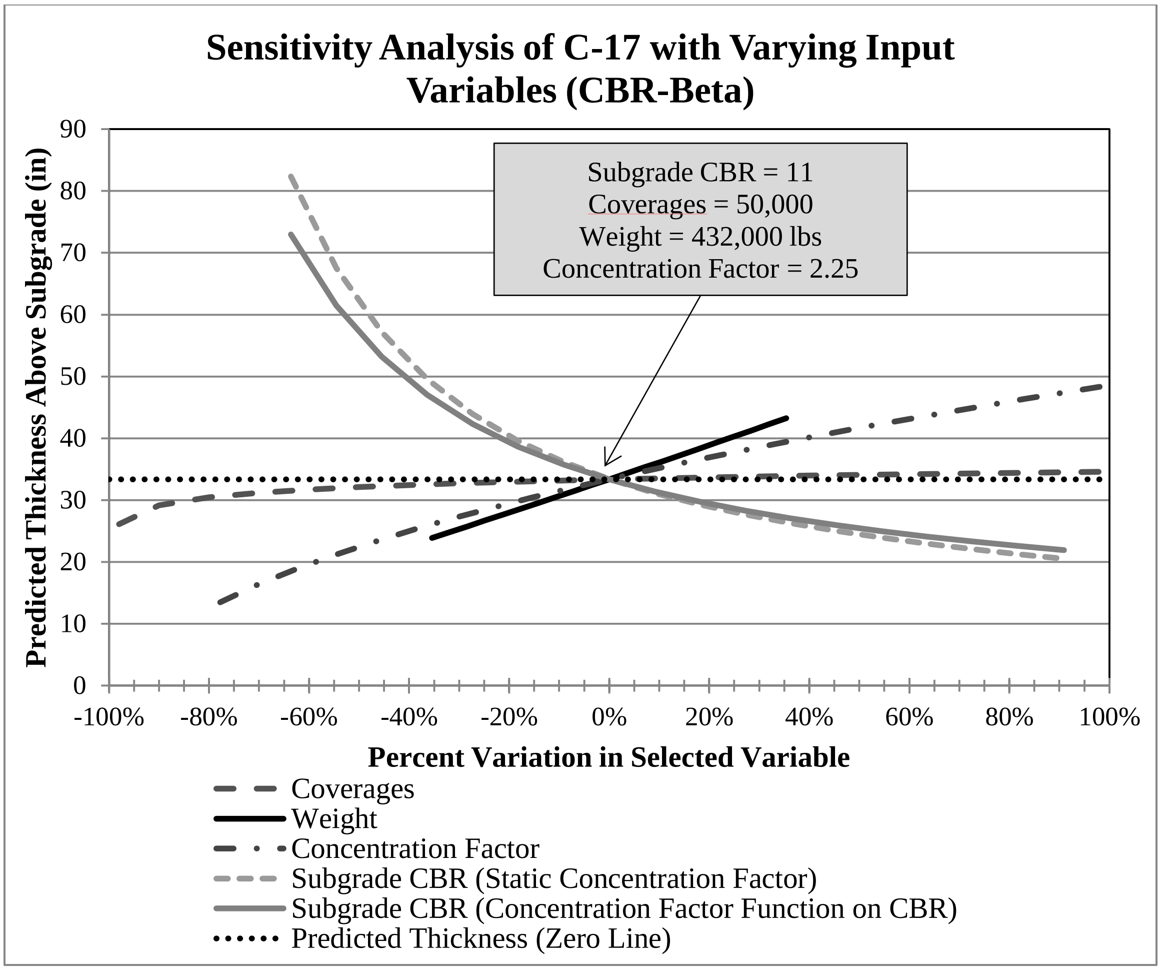

Figure 14. Sensitivity Analysis of C-17 with Varying Input Variables using CBR-Beta

The final formulation of the new design equation resulted in a mechanistic-empirical formulation that is more representative of the subgrade stress (Gonzalez, Barker, & Bianchini, 2012). The ability of the new model to provide better stress predictions and ultimately better pavement designs relies heavily on the concentration factor and β; both variables were empirically calibrated. As with any empirical variable, it is only as representative as the sample it was derived from; additionally it assumes that the sample used to calibrate the variable was representative of the entire population of data. Even with a representative sample, it is still possible to create an empirically defined variable that does not optimally characterize the sample. Examples of this scenario include models that are overly simplistic or fail to incorporate each of the variables necessary to explain the response. This logic is what prompted the evaluation of the current formulation of the concentration factor to determine a more representative model.

Objectives

The initial goal was to evaluate the effect of layer thickness, particularly with stabilized layers, and subgrade depth on the CBR-Beta design method's ability to predict the failure coverages of test section data; however, the focus soon shifted to developing a more predictable concentration factor model. This shift resulted in the necessity to modify the design method to accommodate the test sections with thick base courses and deep subgrades. With only two empirically derived variables, the only variable that could address these issues was the concentration factor. In an effort to address the problem in an organized manner, the overall objective was segmented into portions to build up to the solution in a logical order.

- Identify the current deficiencies in the current concentration factor model by analyzing the model's ability to predict the equivalent thickness and computed concentration factor for historical test section data.

- Determine the commonalities, in terms of loading and pavement structure characteristics, among the test section data points with large variance between the actual and predicted response.
- Identify model formulations, in terms of variables and interaction terms, that most accurately characterize the commonalities; optimize the coefficients for the most representative forms.
- Compare the improved models with the status quo using the median and mean absolute percentage error; quantify the potential improvement of implementing the new model.

Deficiencies with Current Model

The current concentration factor model was developed using single-wheel testing data and was not calibrated for multiple-wheel gear assemblies (Gonzalez, Barker, & Bianchini, 2012). By expanding the empirical data set to 157 data points acquired from various agencies, to include the USACE, the FAA, and Airbus, it was easier to identify inadequacies of the model in its current form; the data used in this research is referenced in Appendix H and included in the references section of this article. As shown in Figure 15, the current model does not accurately fit the computed concentrated factor from 157 data points; the correlation between the data sets is 0.54. Additionally, when the predicted thickness is compared to the equivalent thickness, 30 percent of the data points exceed 0.30 absolute percentage error (APE) and thus skewing the mean score; this is shown in Figure 16. When these points are analyzed as sub-groups, there appeared to be no singular commonality between the points; however, two scenarios appeared to have some influence on the increased absolute percentage error.

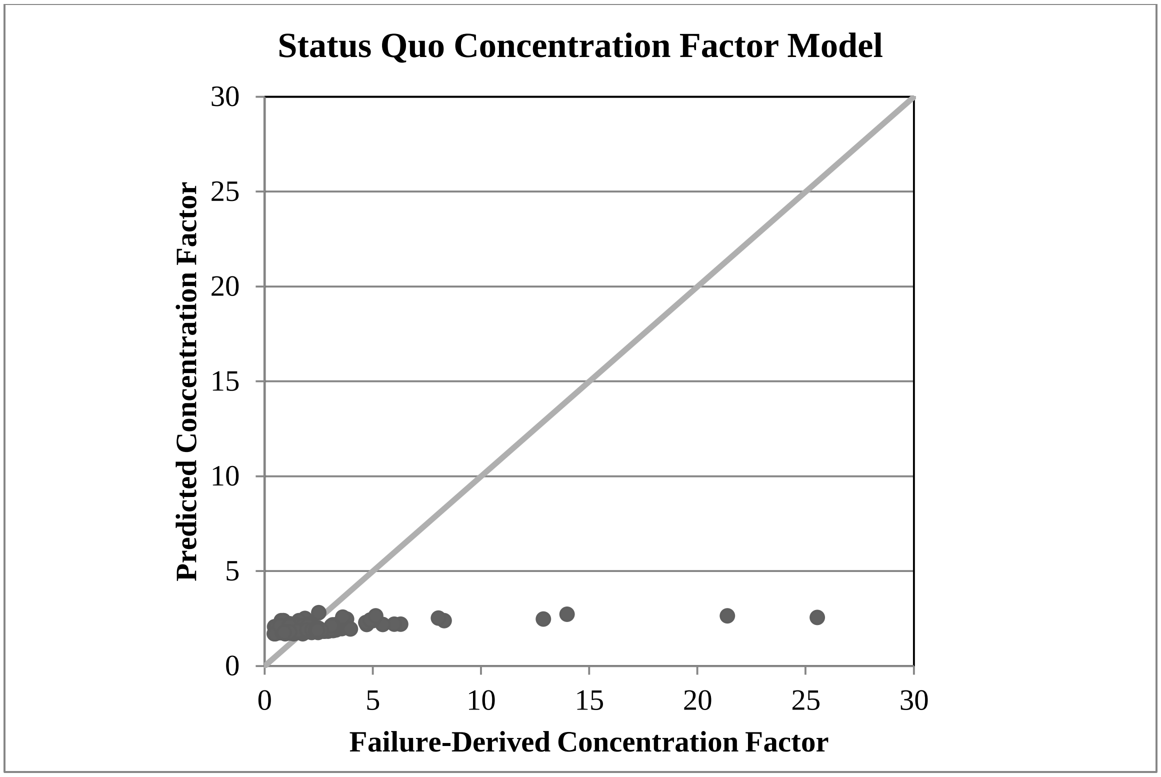

Figure 15. Comparison of the Failure-Derived Concentration Factor and the Predicted Concentration Factor Based on the Status Quo Model (n = 157)

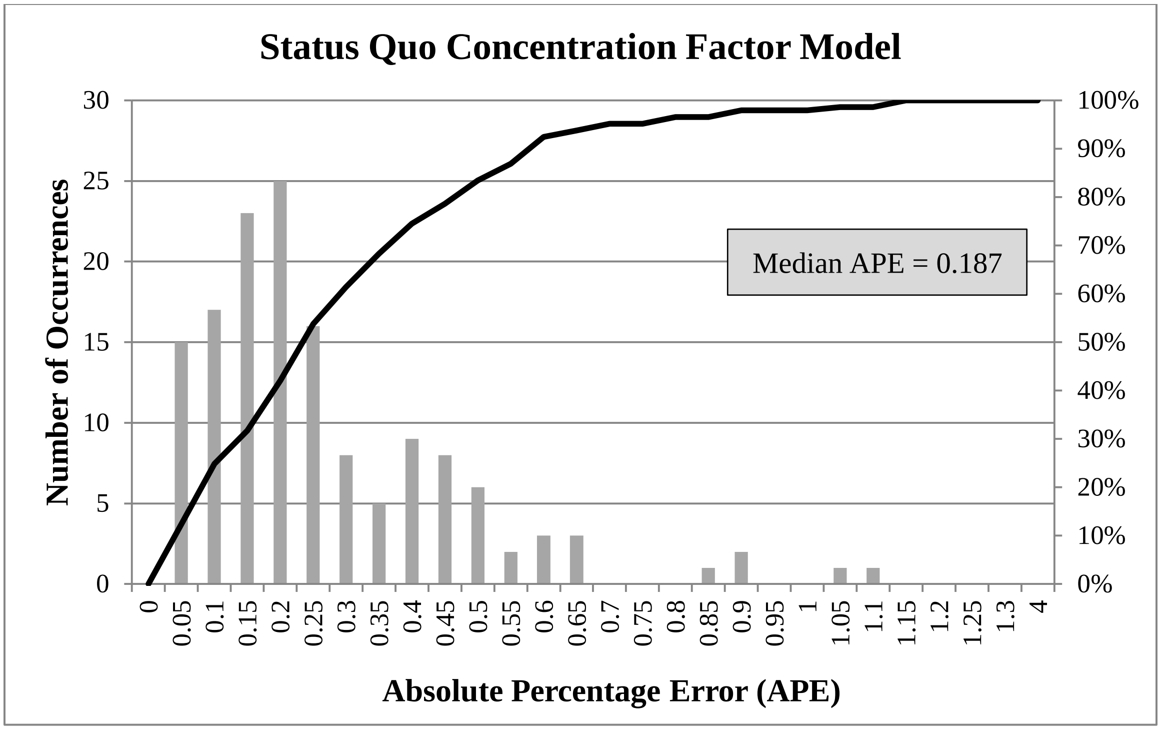

Figure 16. Histogram of Absolute Percentage Error Scores for Predicted and Equivalent Thicknesses Using the Status Quo Concentration Factor Model (n = 157)

Upon investigation of the status quo concentration factor data, a few regions (i.e., a subset of the sample space based on a given set of parameters) of the dataset were observed that appeared to explain much of the elevated APE scores. When the equivalent thickness is less than 30 inches, of the 108 test sections that plot in this region, the current model resulted in 44 percent of these points having APE scores greater than 0.25; comparing all of the test sections with APE scores greater than 0.25, 87 percent plot within this region. The remaining 13 percent of these points are located between the equivalent thicknesses of 32 inches to 48 inches and have ratios of loaded radius to predicted thickness of less than 0.32; this region contained 34 test sections, of which 21 percent had APE scores greater than 0.25. Additionally, for the test sections with base courses in excess of 10 inches, 53 percent had absolute error percentages above 0.25. In terms of gear assemblies in the various test sections, the single-wheel and the 12-wheel gear assembly produced elevated APE scores with mean values of 0.30 and 0.32, respectively; the mean APE of the total model was 0.24.

It was hypothesized that the ratio of predicted thickness to loaded radius, the equivalent thickness, and the base course thickness could explain the majority of the error. To test this theory, these variables were incorporated, along with the subgrade CBR, in the reformulation of the concentration factor model. Since the finalized model would be used for the design of pavements and the equivalent thickness would not be known, the predicted thickness would be substituted into the proposed models in place of the equivalent thickness.

Development of New Models

Since the CBR-Beta design process is an iterative process for multi-wheel aircraft, the optimal concentration factor for each individual test section that would result in the predicted thickness matching the equivalent thickness was determined. Based on the Boussinesq-Frohich theory, formulating the concentration factors using actual vertical stress data would have been ideal; however, the resulting sample size would have been reduced by 74 percent since several of the test samples did not include this data or did not record it. Using the concentration factors, as determined from matching the predicted thickness to the equivalent thickness, it was assumed that the failure of the pavement system occurred when the vertical stress in the subgrade exceeded the allowable stress. The available vertical stress data was used in an attempt to validate this assumption.

With the optimal concentration factors remaining static during the model building phase, the models could be tested outside of the iterative process. This step cut down on the computing time and eliminated the need for certain model formulations. During the initial phases of the model building process, over 40 standard non-linear regression formulations were tested with very limited success. As a result of this initial set back, non-standard models were developed using the variables identified in the previous section in various interactions and cumulative effects.

As an initial starting point, the exponent of 0.1912 was changed into a function of the ratio of predicted thickness to loaded radius multiplied by a constant. With optimized coefficients, this step alone increased the correlation of the computed test section concentration factor to the predicted value from 0.54 to 0.91; the mean absolute percentage error decreased from 0.60 to 0.24. With this success, approximately ten variations of this modified model were developed by including additional additive effects from other variable interactions. From these ten variations, six models were selected to be tested in the iterative CBR-Beta design process and compared against the current model (Equations 13-18):

$$n = 0.60\left(\frac{CBR}{1.83}\right)^{\frac{0.310t}{r}} \tag{13}$$

$$n = 4.73\left(\frac{CBR}{30.35}\right)^{\frac{1.62r}{t}} \tag{14}$$

$$n = 0.96\left(\frac{CBR}{6.40}\right)^{\frac{0.55t}{r}} + 1.24\ln(t) - 3.03 \tag{15}$$

$$n = 0.43\left(\frac{CBR}{1.94}\right)^{\frac{0.35t}{r}} - 1.48\left(\frac{Base\ Course\ Thickness}{Minimum\ Thickness}\right)^{-0.19} + 0.24\ln(t) + 1.12 \quad (16)$$

$$n = 1.61\left(\frac{CBR}{1.74}\right)^{\frac{0.183t}{r}} - 0.43\ln(CBR + 23.72) + 0.245 \quad (17)$$

$$n = 122.71\left(\frac{CBR}{57.68}\right)^{\frac{10.06r}{t}} + 1.21\ln(CBR - 1.46) + 0.41 \quad (18)$$

When these models were incorporated into the CBR-Beta design process, each had boundary issues which caused some of the points not to converge on a solution in the iterative process. It was determined that the cause of this issue stemmed from the concentration factor being a function of the predicted thickness as opposed to being held static using the equivalent thickness of the actual test section. As a general note, when the initial models (Equations 13-18) were entered into the design process with thickness set to equivalent thickness as opposed to the predicted thickness, as would be the case for pavement evaluation, the correlations of the model to the equivalent thickness were all greater than 0.87; some correlations were as high as 0.96.

To fix the boundary issues, the coefficients were manipulated and a constant was included as the last term in Equations 13 and 14. Alleviating the boundary issues proved to be a relatively simple fix by adjusting the coefficients; however, the iterative process caused more complex issues when it came time to optimize the models to minimize the median absolute percentage error. Due to the complexity of the model space, the iterative process, and boundary concerns, traditional optimization tools could not find a minimum solution without reasonable initial estimates. Therefore, the coefficients and constants were manipulated for each model by hand until the median and mean absolute percent errors, as well the visual correlation, were within a reasonable range prior to using an optimization tool. This process was repeated over 50 times per model to ensure the best fit for each model was selected. Inevitably, this method of optimization does not fully alleviate the potential of not finding the optimal solution; however, with 50 trials of varying sign changes and coefficients, the probability that the optimal solution presents a statistically better fit than the initial solutions was reduced.

After correcting the boundary issues and performing the optimization process, it was apparent that the coefficients and constants for each model changed significantly from the static environment (known thicknesses) to the dynamic environment (unknown thicknesses). These changes included significant changes to the magnitudes of the coefficients, as well sign changes. For models 3 and 4 (Equations 15 and 16, respectively), the change in environment required a modification to the interaction of the first term in each model to correct boundary issues and improve accuracy; the exponent referring to the ratio of thickness above subgrade to loaded radius was inversed. The final formulations of the single-layer concentration factor models are presented as Equations 19-24:

$$n = 2\left(\frac{CBR}{6}\right)^{\frac{0.1t}{r}} + 0.5 \tag{19}$$

$$n = 2\left(\frac{CBR}{6}\right)^{\frac{2r}{t}} + 0.3 \tag{20}$$

$$n = 2\left(\frac{CBR}{6}\right)^{\frac{1.5r}{t}} + 0.10\ln(t) - 0.18 \tag{21}$$

$$n = 2\left(\frac{CBR}{6}\right)^{\frac{1.25r}{t_{total}}} + 0.15(t_{ac+base})^{0.50} - 0.25\ln(t_{total}) + 0.40 \tag{22}$$

$$n = 2\left(\frac{CBR}{6}\right)^{\frac{0.15t}{r}} + 0.1\ln(CBR + 20) + 0.4 \tag{23}$$

$$n = 2\left(\frac{CBR}{7}\right)^{\frac{1.8r}{t}} + 0.6\ln(CBR + 20) - 1.6 \tag{24}$$

Comparison of Single-Layer Models with Status Quo

As shown in Figure 17, the status quo concentration factor model produced increased levels of relative error when the equivalent thickness was between 20 and 50 inches. This range contributed to the elevated maximum APE score of 1.07; however, the median score was 0.19 with an R^2 score of 0.66. Using these scores as the metric for analysis, along with the sum of the squared error (SSE) and the mean and median difference between the predicted thickness and the equivalent thickness (referred to as Δ; positive values reflect overdesigned pavements), the research compared each of the six models against the status quo. The results of this analysis are summarized in Table 13.

Four of the proposed models outperformed the status quo as measured by the various statistical measures. For example, Model #4 (Equation 22) produced a three percent increase in the R^2 value relative to the status quo with the current equivalency factors. Comparing the SSE values, Model #4 improved on the status quo by ten percent with the current equivalency factors. The three other models saw similar improvements albeit at lesser magnitudes, while Model #1 (Equation 19) and Model #5 (Equation 23) appeared to provide no improvement over the status quo.

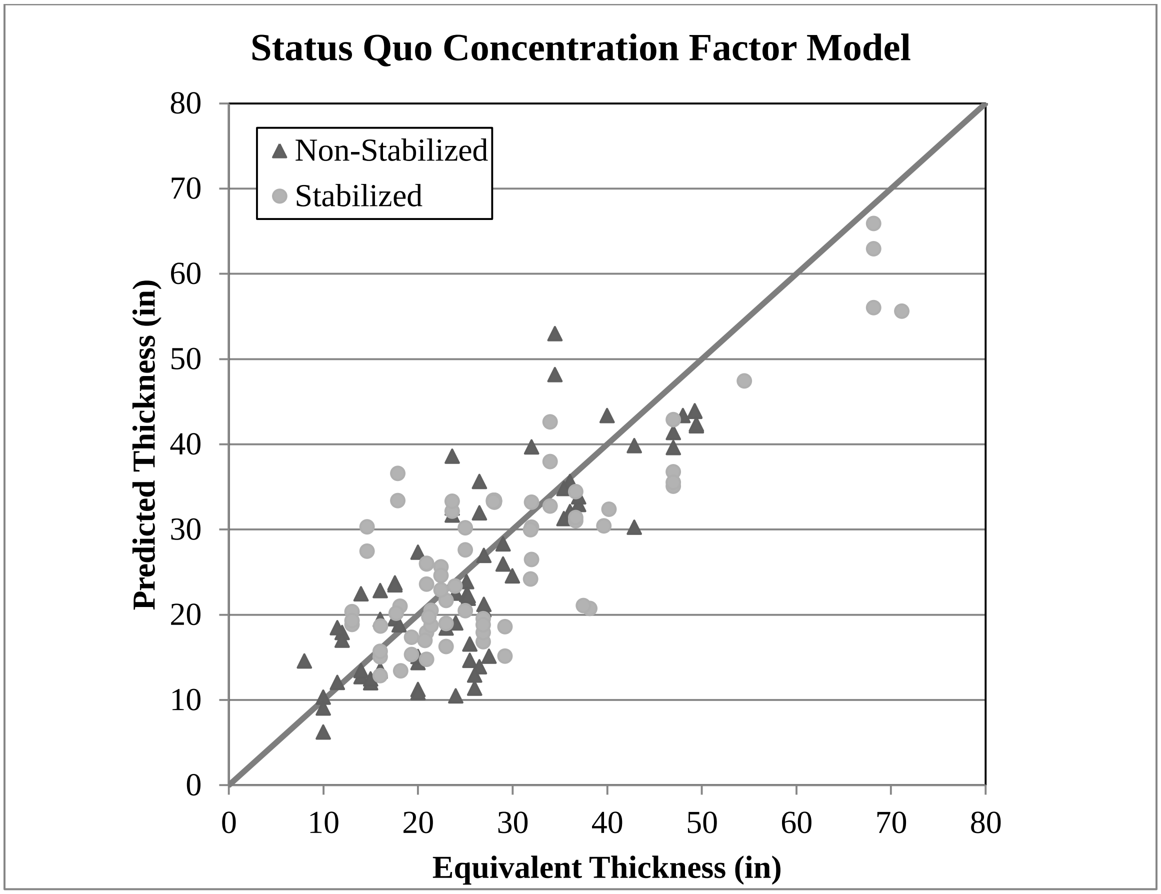

Figure 17. Comparison of the Equivalent and Predicted Thicknesses for the Status Quo Concentration Factor Model (n = 157)

Table 13. Comparison of Predicted Thickness to Equivalent Thickness for the Six Concentration Factor Models – Current Equivalency Factors

Current Equivalency Factors (n = 157)							
	Status Quo	**Modified Model #1**	**Modified Model #2**	**Modified Model #3**	**Modified Model #4**	**Modified Model #5**	**Modified Model #6**
R^2	0.66	0.64	0.67	0.68	0.68	0.62	0.67
SSE	8,085	9,611	8,038	7,661	7,278	10,641	7,679
MAPE	0.25	0.28	0.27	0.25	0.24	0.30	0.26
MdAPE	0.19	0.20	0.19	0.20	0.21	0.22	0.19
Mean Δ	-0.69	2.13	0.82	-0.24	-0.64	2.25	-0.10
Median Δ	-1.26	1.08	0.82	-0.24	-0.64	2.25	-0.10
Std Dev Δ	7.46	7.89	7.32	7.26	7.11	8.00	7.28

Formulation of Two-Layer Models

To further refine the models, potential improvements that could be garnered from deriving different concentration factors for the subgrade and the subbase for use with the CBR-Beta design method were investigated. This idea was first suggested by Bianchini (2014) in which she was able to derive a multiple linear regression model to predict the concentration factor for the subgrade to calculate the vertical stress on the subgrade for flexible pavement systems with known layer thicknesses. This model was initially used, but significant boundary issues were observed when the layer thicknesses were unknown and the concentration factor model was used to determine the thicknesses. Based on the initial promise of her work, the concept was investigated further using non-linear models similar to the formulations in Equations 19-24. Additionally, the two-layer concept theoretically appeared to replicate the pavement structure system more than a single-layer model, as the single-layer model assumes homogeneity, which does not accurately characterize the superior strength and load distributing properties of the wearing and base courses.

Following a methodology similar to the one used for the single-layer models of the subgrade, the experimental concentration factor of the subbase was determined. For test sections with a base course over a subbase, the subbase was assumed to have a CBR of 20 percent and the equivalent thickness of the wearing and base courses were calculated; for test sections without a subbase course, the test section characteristics remained unchanged. Starting with the models presented in Equations 19-24, the coefficients and constants were optimized to minimize the median APE of the models. After several attempts, a model that resulted in a correlation of at least 0.80 with the interactions from the subgrade models (Equations 19-24) could not be found. As a result, the interactions from the single-layer models were varied. After several rounds of testing different interactions, Equation 25 produced a median APE of 0.25 and a correlation of 0.82. To pair the subbase model with the subgrade model, the subbase concentration factor was incorporated into subgrade model #4 (Equation 22), which appeared to be the most statistically accurate way of predicting the equivalent thickness. After optimizing the constants and coefficients of the subgrade model with the new interaction, the resulting model is shown as Equation 26.

$$n_{SB} = 2\left(\frac{CBR_{SB}}{20}\right)^{\frac{1.5r}{t_{ac+base}}} + 0.2\ln(t_{ac+base}) - 0.5\sqrt{\frac{t_{ac}}{4}} + 0.5 \qquad (25)$$

$$n_{SG} = 2.25\left(\frac{CBR_{SG}}{7}\right)^{\frac{1.5r}{t_{total}}} - 0.2(t_{ac+base})^{\frac{0.5}{n_{SB}}} - 0.05\ln(t_{total}) + 0.5 \qquad (26)$$

where,

t_{total} = *Thickness of Above Subgrade*
$t_{ac+base}$ = *Thickness of Above the Subbase (Wearing and Base Courses)*

t_{ac} = *Thickness of the Wearing Course*
r = *Contact Radius*
CBR_{SB} = *California Bearing Ratio of the Subbase Material*
CBR_{SG} = *California Bearing Ratio of the Subgrade Material*
n_{SB} = *Subbase Stress Concentration Factor*
n_{SG} = *Subgrade Stress Concentration Factor*

Using the two-layer model formulation, a mean and median APE of 0.22 and 0.17, respectively, were calculated when predicting equivalent thickness. With each of the previous single-layer models, the maximum APE score exceeded 1.03; however, with the two-layer formulation, the maximum value was reduced to 0.94. Additionally, the R^2 value for this model was 0.70, which signified a 6.1 percent improvement over the status quo model. The results of this model are shown as Figure 18.

Analyzing the two-layer concentration factor model based on a comparison of the mean APE values separately for stabilized and non-stabilized soils, stabilized soils demonstrated an increased rate of percentage error. This indicated that with revised equivalency factors, the two-layer model, and for that matter the other models as well, could be improved in terms of statistical measures of fit. As this is outside of the scope of this research, this refinement was identified for future analysis.

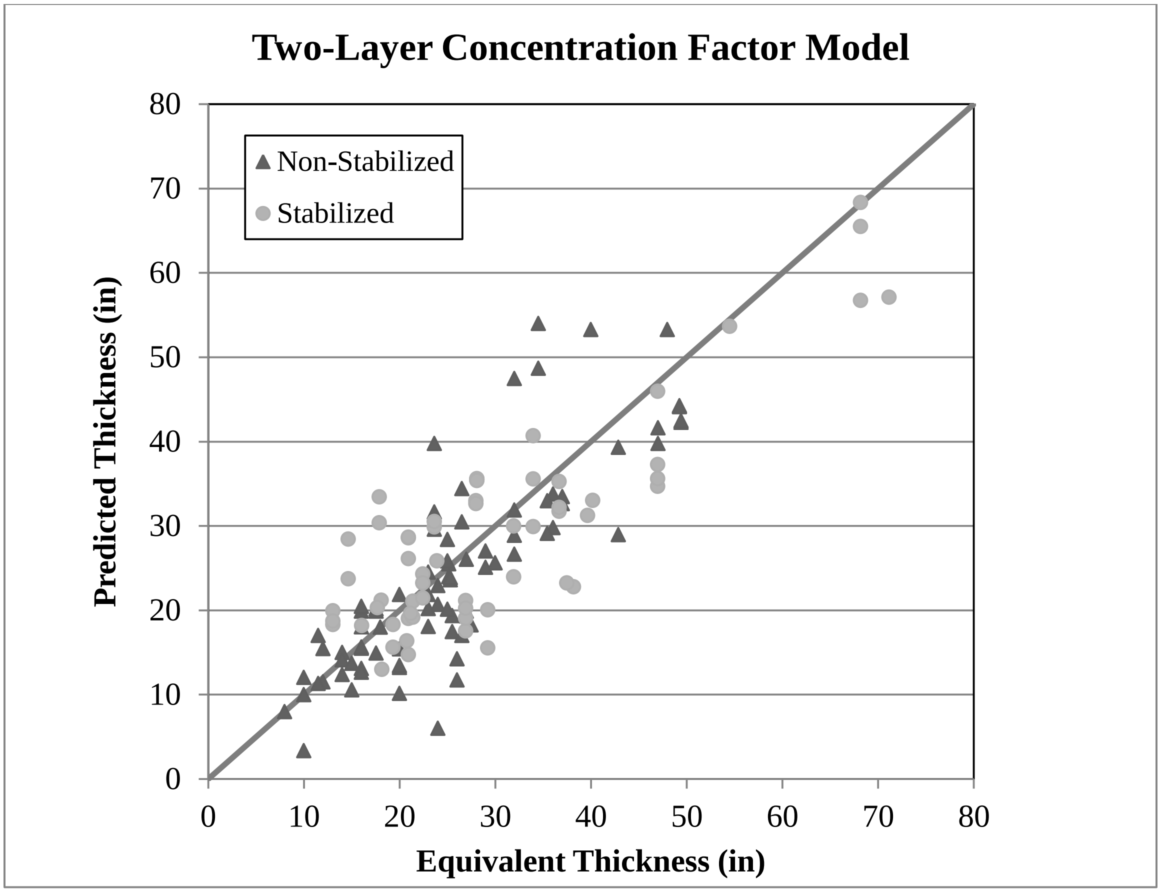

Figure 18. Comparison of Equivalent and Predicted Thickness for the Two-Layer Concentration Factor Model with Current Equivalency Factors (n = 157)

Analysis of Results

To compare the results from the single-layer and two-layer model concepts, the data for the models with the highest statistical measures were compared to the status quo model. As shown in Table 14, both the two-layer and single-layer models resulted in higher R^2 values and lower median APE scores; however, the two-layer model construct appeared to provide the highest degree of predictability in terms of accurately predicting the equivalent thickness. This statement was further supported by evaluating the SSE values, in which the scores for the two-layer models are reduced relative to the other models by over four percent.

Table 14. Comparison of Status Quo, Single-Layer, and Two-Layer Concentration Factor Models (n = 157)

	Current Equivalency Factors		
	Status Quo	**Model #4 Single-Layer**	**Two-Layer Model**
R^2	0.66	0.68	0.70
SSE	8,248	7,407	7,766
MAPE	0.24	0.24	0.22
MdAPE	0.19	0.19	0.17
Mean Δ	-0.76	-0.08	-0.53
Median Δ	-1.34	-0.54	-1.02
Std Dev Δ	7.23	6.89	7.04

The main drawback with the two-layer model was the negative mean and median Δ values. These scores become problematic when one considers that the negative Δ values correspond to under-designed pavements. This realization coupled with the mean and median scores both resulting in negative values implied that the majority of the pavements, approximately 58 percent, resulted in negative Δ values. Typically these scores would be a cause for concern; however, with significantly low mean and median APE values overall, the median score for test sections with negative Δ values is approximately 0.146. This low median APE score appeared to indicate that although the model tended to under-predict equivalent thickness, the model was still relatively accurate at predicting the actual value. With improved equivalency factors and further optimized coefficients and constants, it was assumed that the mean and median values would shift upwards closer to zero. This improvement would inevitably shift the cumulative distribution of the APE scores for the two-layer model (as shown in Figure 19) further to the left with a median closer to zero relative to the other models.

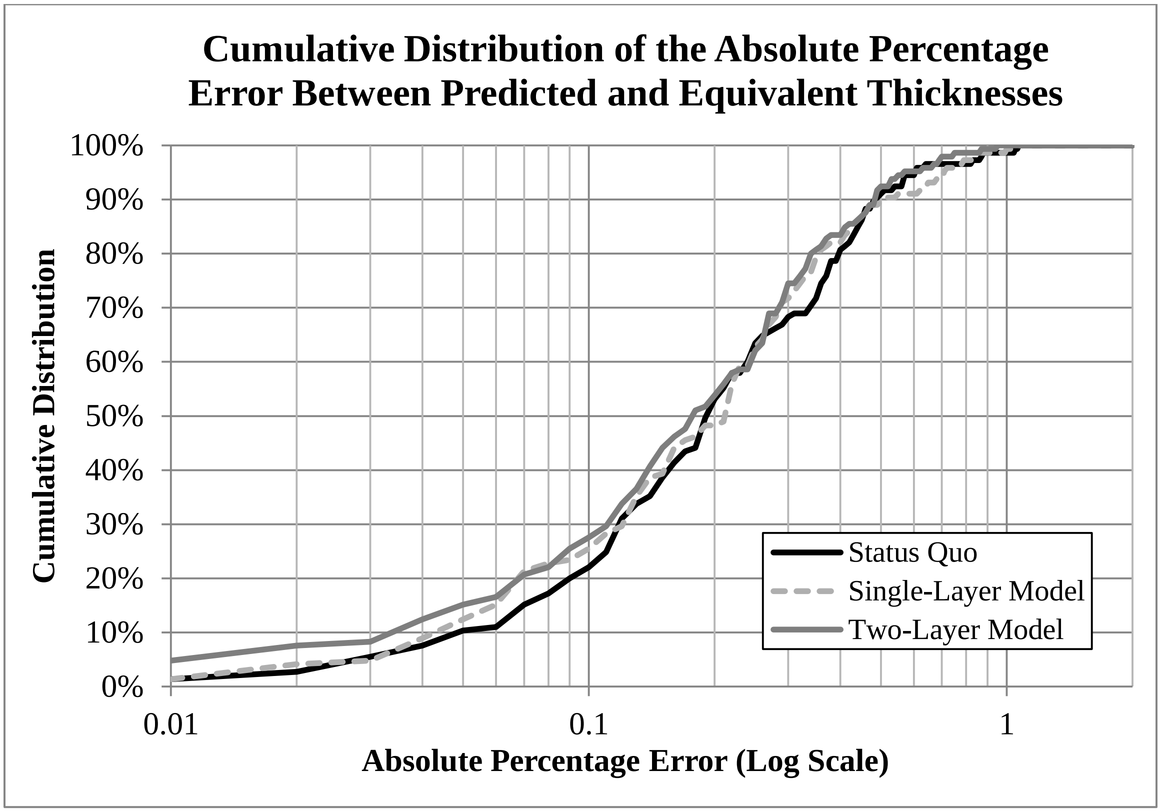

Figure 19. Cumulative Distribution of the Absolute Percentage Error Between the Predicted and Equivalent Thicknesses

Comparison with Actual Stresses

As shown in Figure 20, the comparison between the actual subgrade vertical stress and the prediction from the two-layer model included a high-degree of disparity between the predicted and the measured stress. The predicted stress values were calculated at the equivalent depth of the measurement device in the pavement system that the model was attempting to represent. However, when the model was allowed to iterate the depth of the subgrade to match the predicted stress to the allowable stress, the model was able to produce predicted thicknesses with a correlation of 0.88 to the actual equivalent thicknesses. This contradiction between the high variance with regard to the vertical stress and the high degree of fit with regard to the equivalent thickness lent itself to identifying the allowable stress criteria (see Equation 11) as the casual factor. As a general note, the status quo concentration factor model produced similar results for the vertical stress, but with a slightly less degree of fit for the equivalent thickness plot.

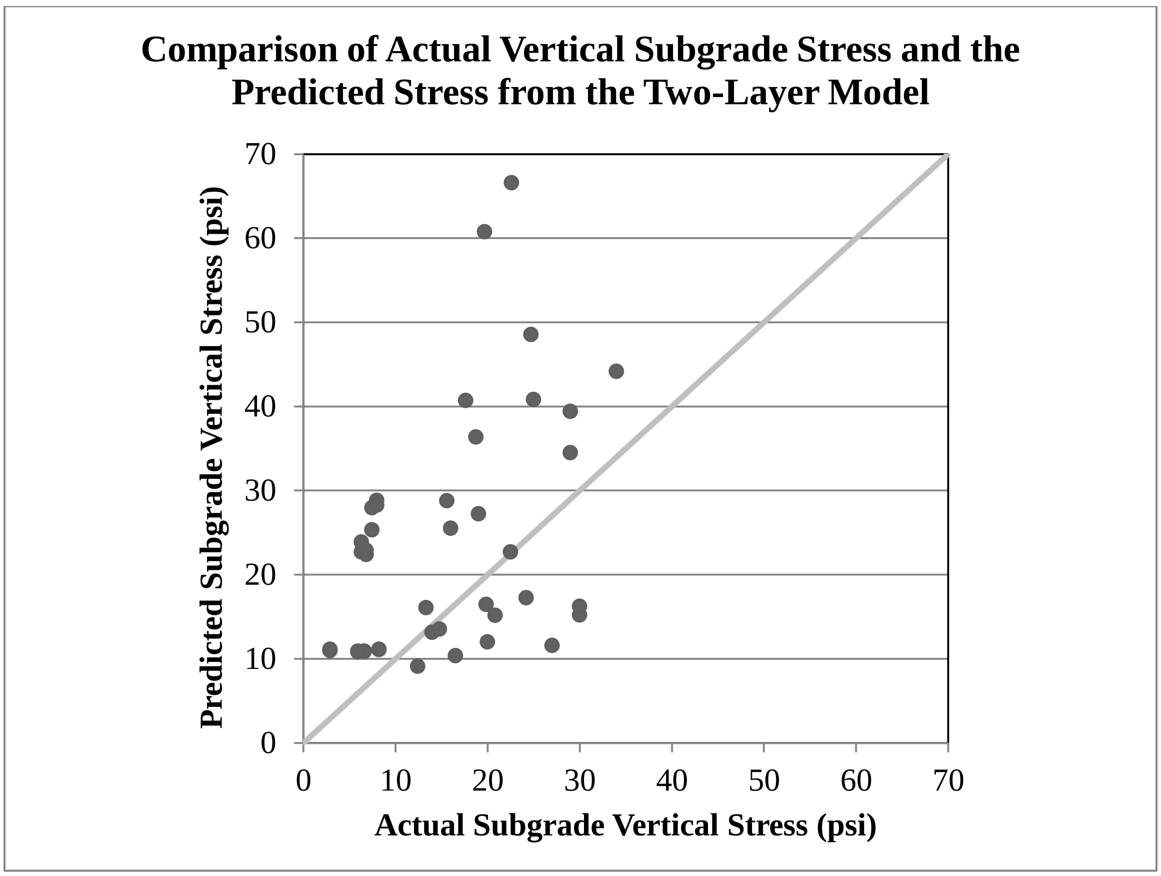

Figure 20. Comparison of Actual Vertical Subgrade Stress and the Predicted Stress from the Two-Layer Concentration Factor Model at Equivalent Depth of Subgrade (n = 41)

To further support the previous statement regarding the allowable stress criteria being the casual factor, the concentration factor was reverse-calculated for the 41 test sections based on the measured vertical stress at the equivalent depth of the measurement device. Using the stress-derived concentration factor for each test section, the CBR-Beta process was iterated to the subgrade depth, with a fixed concentration factor, to determine the thickness above the subgrade necessary to equate the predicted vertical stress and the allowable stress. If the allowable stress was reflective of the measured stress, then the predicted thickness would closely match the equivalent thickness. However, as shown in Figure 21, the predicted thickness does not appear to represent the equivalent thickness, as further supported by the 0.42 correlation between the two variables.

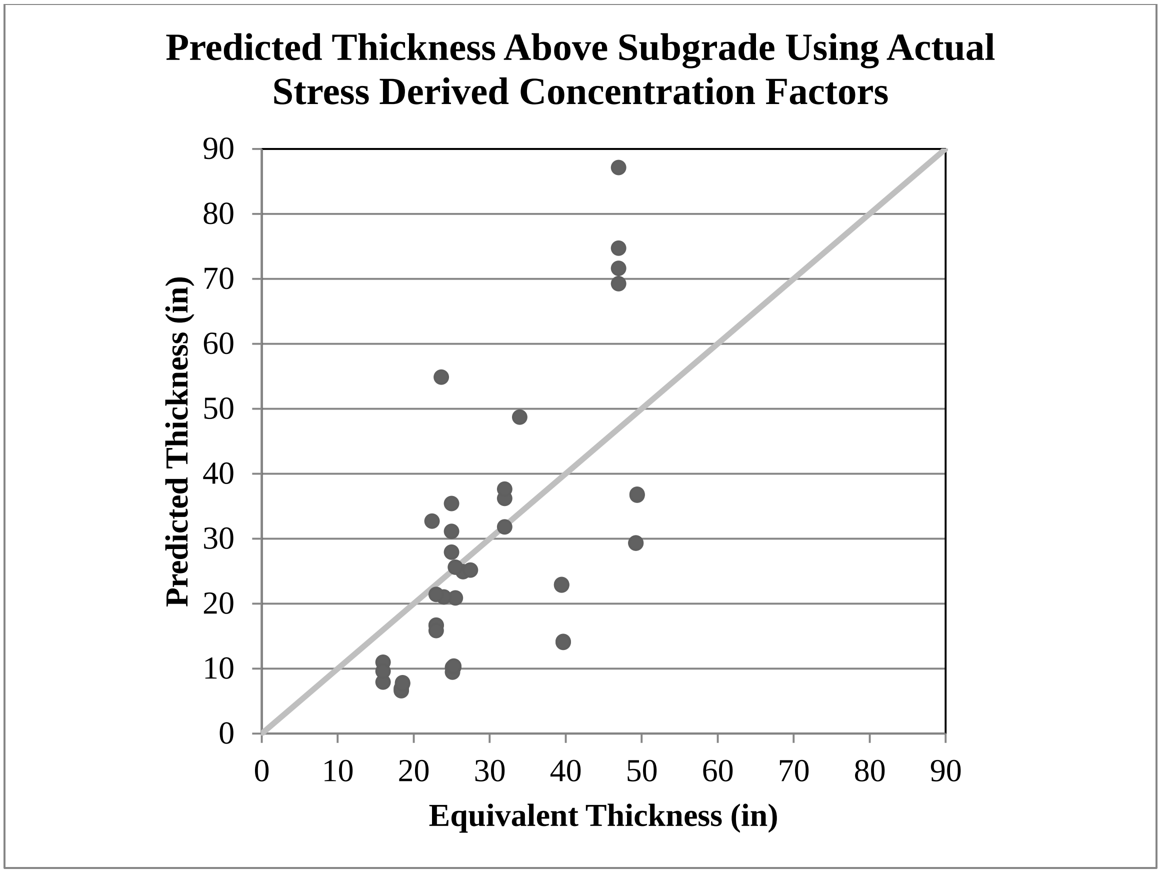

Figure 21. Comparison of Equivalent and Predicted Thicknesses with Concentration Factors Derived Using Actual Subgrade Vertical Stresses (n = 41)

Investigating this concept further, the rutting failure defined concentration factors were compared with the vertical stress defined factors to identify trends. As shown in Figure 22, there appeared to be a highly correlated segment of the population, as identified by the dashed ellipse, that plot just below the 1:1 sloped line. Due to the high degree of correlation between this segment of the sample space, it was assumed that with a change in the allowable stress criteria, this segment would plot closer to the 1:1 sloped line, thereby, producing allowable stresses that more closely match the measured vertical stress.

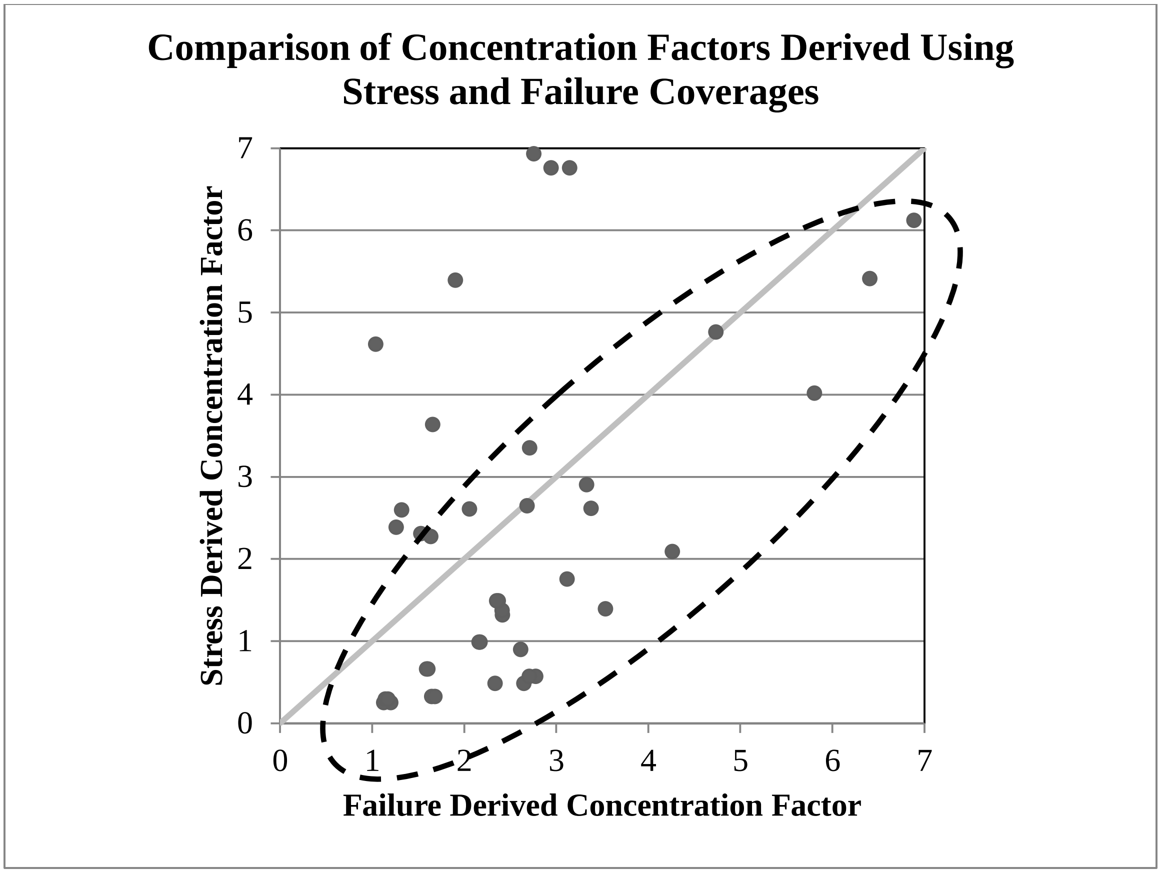

Figure 22. Comparison of Concentration Factors Derived Using Stress and Failure Coverages – Current β Formulation (n = 41)

The allowable stress criteria is a function of CBR and β; therefore, these two variables control the discrepancy between the measured and allowable stress, assuming the formulation of the criteria remains the same. Since CBR is a material property, it cannot be modified. On the other hand, β is an empirically derived formulation as a function of coverages and can be adjusted to reduce the discrepancy between the actual and measured stresses. Modifying the β criteria was beyond the scope of this research; however, as a proof of concept, the β criteria was optimized in its current form to verify the assumption that β was the source of the discrepancy between the stress values. As shown in Figure 23, with modified β criteria, the stress and failure defined concentration factors appeared to track more closely together.

As shown in Figure 23, and visually apparent in other figures, the data used for this analysis contained several outliers. Rather than remove the outliers, it was decided to leave these test sections in the analysis in an effort to analyze how the different models would handle these points. Additionally, without further information as to the reason for the outlier behavior, there was no reason to assume these test sections were not part of the population data set.

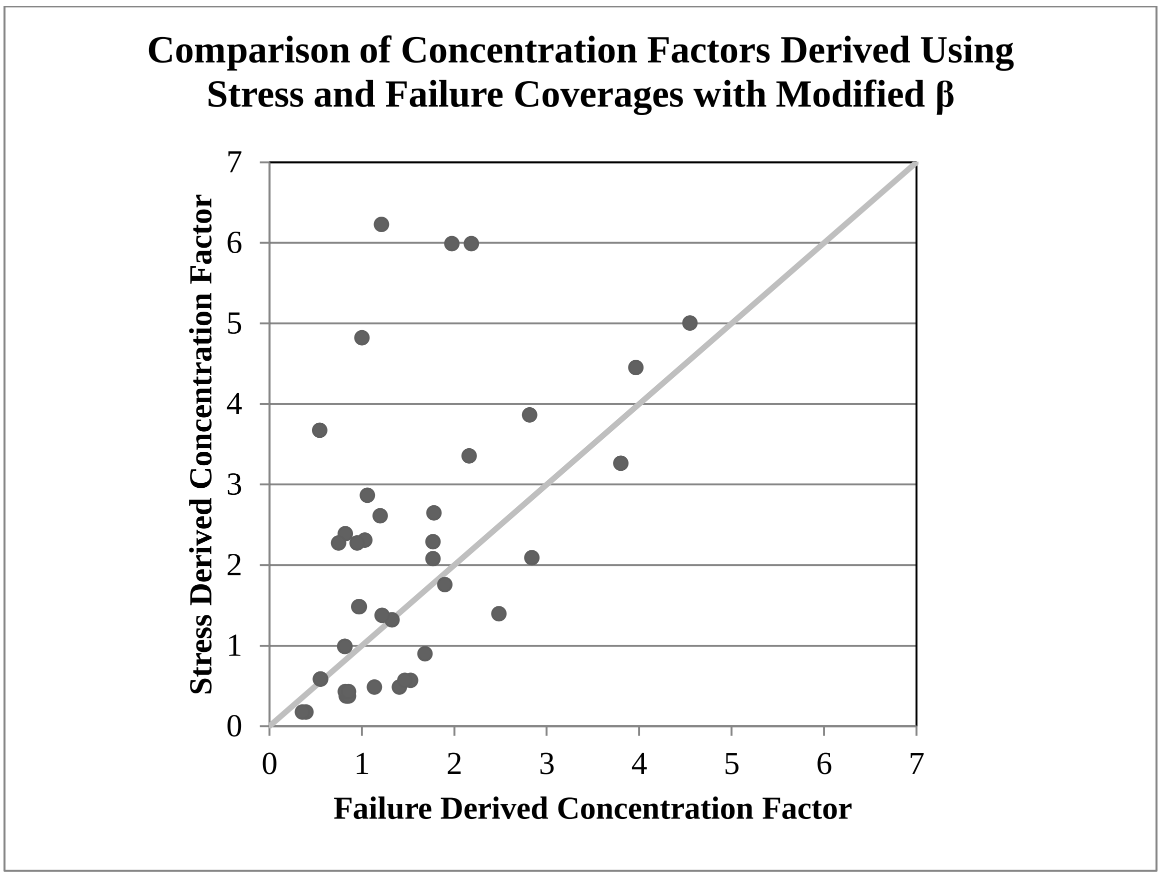

Figure 23. Comparison of Concentration Factors Derived Using Stress and Failure Coverages – Modified β Formulation (n = 41)

Summary and Conclusions

Based upon the results of the comparison between the actual stresses and the calculated stresses, it appeared that the empirically derived nature of the concentration factor and subsequent application created a discrepancy with the theoretical intent of the variable. In other words, the status quo concentration factor and the proposed formulations of the model presented herein create variables that rely heavily on the accuracy of the other empirically derived variable in the CBR-Beta design methodology, β. Errors in either model can potentially influence the other variable, particularly during the calibration stages of the design method in which the predicted thicknesses are matched to the equivalent thicknesses. However, based on some initial work during this research to modify the β criteria and align the failure and stress derived concentration factors, it was recommended that further investigation be conducted to identify the optimal model for aligning the two factors. By aligning the concentration factors, the degree to which β influences the concentration factor formulation, particularly with the incorporation of the iterative depth concept as proposed herein, will be significantly reduced. In doing so, the concentration factor can more accurately reflect its theoretical intent in calculating various stress and strain values, to include the vertical stress on the subgrade.

As mentioned previously, due to limitations with optimization tools, the two-layer model could be further improved for design if the coefficients and constants could be further optimized while avoiding the boundary issues associated with the iterative process. Even with the two issues regarding β and optimality, the two-layer concentration factor model, as proposed herein or with slight modifications, provided a higher degree of predictability over the status quo. Due to the boundary conditions and the multiple iterative dimensions involved with the unknown layer thicknesses, the design offers slight improvements; however, with known layer thicknesses, as in the situation with pavement evaluation, the two-layer formulation improves the R^2 relative to the status quo model by 20 percent. This improvement is shown below in Table 15.

Table 15. Comparison of Status Quo and the Two-Layer Concentration Factor Models for Design and Evaluation (n = 157)

	Current Equivalency Factors		
	Status Quo	**Two-Layer (Design)**	**Two-Layer (Evaluation)**
R^2	0.66	0.70	0.79
SSE	8,248	7,766	5787
MAPE	0.24	0.22	0.17
MdAPE	0.19	0.17	0.13
Mean Δ	-0.76	-0.53	-1.32
Median Δ	-1.34	-1.02	-1.19
Std Dev Δ	7.23	7.04	5.95

Overall, the conclusions of this research appear to support the assertion that the CBR-method's assumption of a homogenous layer is conservative. Using a two-layer model provides a more accurate representation of the additional stiffness and strength of the layers above the subgrade, particularly the wearing and base courses. Further research is necessary to continue to refine the proposed concentration factor models to address boundary issues and further optimize the models.

Disclaimer

The views expressed in this article are those of the author and do not reflect the official policy or position of the United States Air Force, Department of Defense, or the United States government.

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VI. White Paper: Applicability of the CBR-Beta Design Methodology for Highway Pavements

This chapter contains the white paper developed for the U.S. Air Force Civil Engineer Center (AFCEC) in response to a request regarding the ability of the CBR-Beta design method to model highway pavements. The goal of the white paper was to investigate the potential for using highway testing data to bolster the airfield testing database for later empirical uses. Additionally, the white paper proposes an extension of the CBR-Beta method for highway pavement design and evaluation.

Applicability of the CBR-Beta Design Methodology for Highway Pavements

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Introduction

At the request of the Air Force Civil Engineer Center (AFCEC), the potential of the CBR-Beta design method to leverage the more abundant highway pavement test section data was investigated in an effort to bolster the size of the database used to derive the empirical variables in the CBR-Beta model. At the time of the request, the experts in the conference call were split on this possibility (Personal Communication, 30 Aug 2013). With a significant difference between airfield and highway loads, to include the failure or allowable stress/strain criteria, the thought was that the model could not accurately characterize the loads. On the other hand, due to CBR-Beta's reformulation that made the model more mechanistic than its predecessor, the logic holds that mechanistic processes would remain unchanged; however, the empirical variables could casue issues. The investigation presented in this chapter attempted to answer this question using a sample of conventional, non-stabilized highway test sections.

Objectives

There is an abundance of texts that describe the differences between highway and airfield flexible pavements, to include the applied loads; however, very few sources exist that discuss using airfield design methods for highway pavements. The lone venture into the topic that could be found was undertaken by Jacob Uzan in 1985, as he offered a modification to the load-repetition factor of the CBR-Alpha criteria to model highway pavements. This modification was necessary in his opinion to account for the higher number of coverages experienced by highway pavements; the airfield criteria were not calibrated at this high level of coverages, so a modification was necessary to define this region (Uzan, 1985). His primary source of test section information came from the American Association of State Highway Officials (AASHO) road test in the mid-to-late 1950s. It was with the success seen by Uzan's (1985) study and the mechanistic reformulation of the CBR procedure that this study sought to build off. As such, the primary objectives of this study were to assess the ability of the CBR-Beta method to model highway pavements and to evaluate the carry-over potential of the highway data to airfield data.

Research Methods

As this was the first study of its kind into highway pavements with the CBR-Beta procedure, this study focused on initial feasibility using non-stabilized test sections only. With non-stabilized test sections, a control study was compared to the non-stabilized airfield test sections; this alleviated potential issues with dealing with stabilized soils. Using mean and median absolute percentage error (MAPE and MdAPE, respectively), it was assumed that if the CBR-Beta could accurately model highway pavements, then the scores for these statistical measures of fit would result in equivalent or better scores relative to the same metrics for the airfield test sections.

The data used for this study came from two sources: (1) the AASHO road test and (2) the Minnesota Road Research Facility (MnROAD). Both of these tests contained multiple configurations of conventional highway pavements and were conducted in a test track manner, as opposed to the linear heavy-vehicle simulator typically used for airfield testing (Minnesota Department of Transportation, 2012; Highway Research Board, 1962a; Highway Research Board, 1962b). The MnROAD test report recorded the passes for the two different vehicles in terms of equivalent single-axle loads (ESALs); therefore, the AASHO road test sections had to be converted using the ESAL conversion chart published in the Asphalt Institute's *MS-1: Thickness Design* manual (Asphalt Institute, 2008). This conversion allowed the inputs to be standardized; however, it inevitably added some additional variability into the model, as the ESAL conversion factors were empirically derived.

Direct Application of Current CBR-Beta Method

As shown in Figure 24, the highway test sections, when plotted as equivalent thickness against predicted thickness, demonstrate a relatively low correlation; however, when overlaid against the airfield test sections, all the highway test sections plotted within the variances seen for the airfield sections, although the mean absolute error was more negative for the highway data. Analyzing the graph further, the highway test sections from the two sources plotted in a segregated manner in that the AASHO road test sections plotted together in the upper grouping of highway data within Figure 24. The disparity between the two groupings was assumed to be the result of the extreme difference in subgrade CBR values. For the AASHO sections, the subgrade CBR was fixed at three percent; whereas, the MnROAD sections varied subgrade CBR between 7.3 to 15.4 percent.

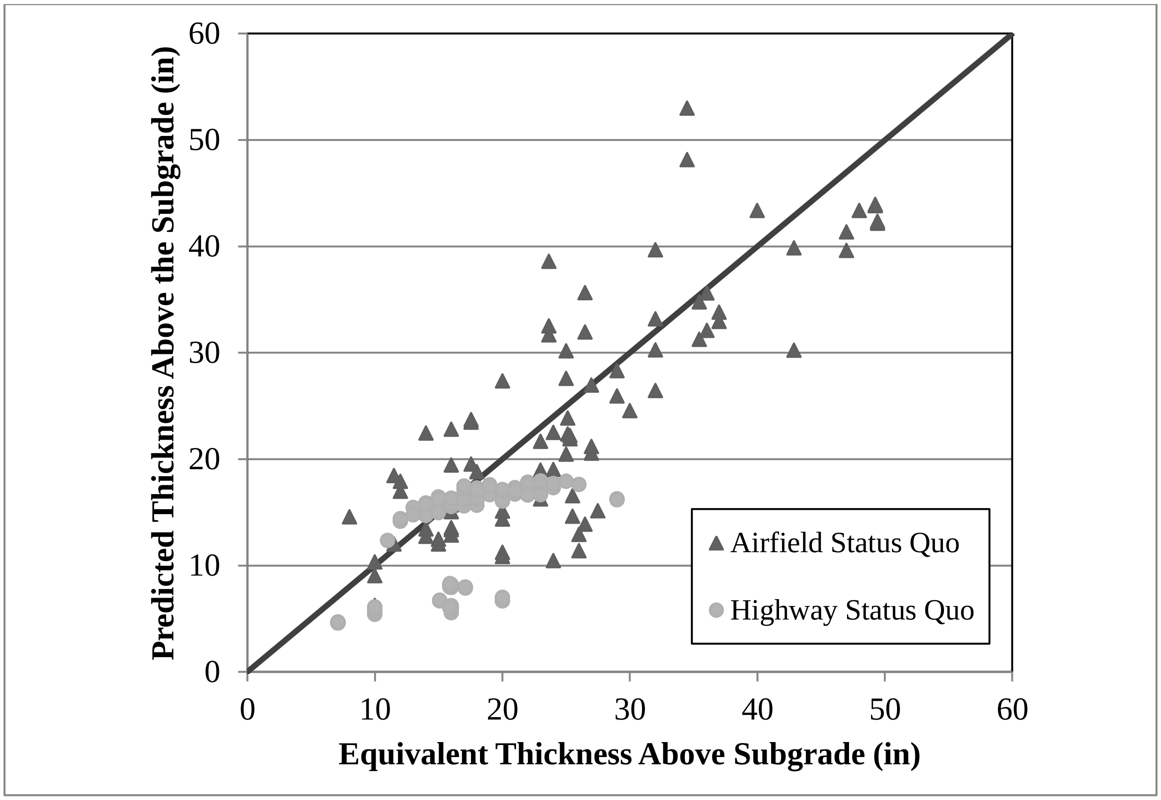

Figure 24. Comparison of Equivalent and Predicted Thicknesses Above the Subgrade for Non-Stabilized Highway and Airfield Pavements with Status Quo Concentration Factor Model (n = 82 – Airfield; n = 123 – Highway)

Analyzing the cumulative distributions for the absolute percentage error (APE) as shown in Figure 25, it was confirmed that the highway data contained a higher rate of error, as the highway distribution lagged behind the airfield distribution. For comparison purposes, the MAPE was 25 percent higher for the highway data. The failed Wilcoxon Rank-Sum Test supported both of these points, leading to the conclusion that CBR-Beta with the status quo concentration factor would produce higher error rates compared to airfield data. This higher rate of error would prove problematic in an effort to utilize highway test sections for empirical supporting data.

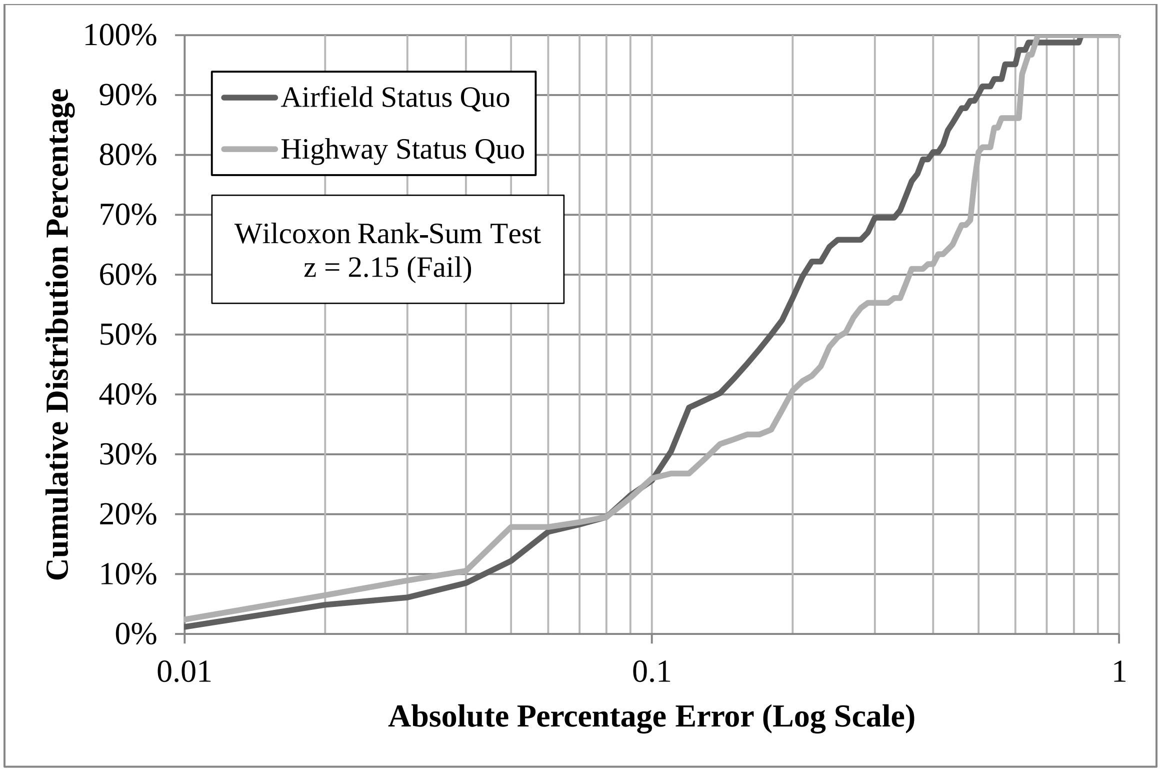

Figure 25. Comparison of the Cumulative Distributions of the Absolute Percentage Error Values for Highway and Airfield Pavements using CBR-Beta with the Status Quo Concentration Factor Model (n = 82 – Airfield; n = 123 – Highway)

Modeling the highway pavements with the two-layer concentration factor model determined in Chapter V was also considered; however, the model experienced significant boundary issues causing several test section predictions to not converge in the iterative CBR-Beta design process. With this failure to directly apply airfield criteria to highway pavements, the focus shifted to customizing the CBR-Beta design method specifically for highway pavements. This customization had to be accomplished by redeveloping the empirical formulations for both β and the concentration factor.

Customized Two-Layer Model

Highway and airfield pavements utilize different failure criteria, a half inch and one inch surface rutting, respectively; therefore, logic dictated that the allowable stress or strain criteria would be significantly different (Mallick & El-Korchi, 2013). To recalibrate β for highway pavements, the Asphalt Institute's subgrade strain criteria (Equation 27) were used as a baseline and the coefficients were adjusted in the airfield β formulation (Equation 28) to fit the strain criteria (Janoo, Irwin, & Haehnel, 2003):

$$N_d = (1.365 \times 10^{-9})(\varepsilon_c)^{-4.447} \tag{27}$$

$$\log \beta = \frac{1.5441 + 0.0730 \log(Coverages)}{1 + 0.2354 \log(Coverages)} \tag{28}$$

where,

N_d = Number of Passess
ε_c = Subgrade Strain
β = Function of Subgrade Stress and Coverages
Coverages = Number of Coverages

After equating compressive subgrade strain in terms of β, this relationship into Equation 27, which was then solved the equation in terms of β. This manipulation resulted in the formulation of the Asphalt Institute's subgrade strain criteria in terms of β presented as Equation 29.

$$\beta = 1500\pi \left(\frac{5.1036(Coverages)}{1.365 \times 10^{-9}}\right)^{\frac{1}{-4.447}} \tag{29}$$

Using Equation 29, the coefficients of the airfield β formulation were modified to align the new highway β curve with the subgrade stain criteria. As shown in Figure 26, the highway β curve (Equation 30) matched the strain criteria with a 100 percent R^2 score.

$$\log \beta = 1.5206 - 0.2249 \log(Coverages) \quad (30)$$

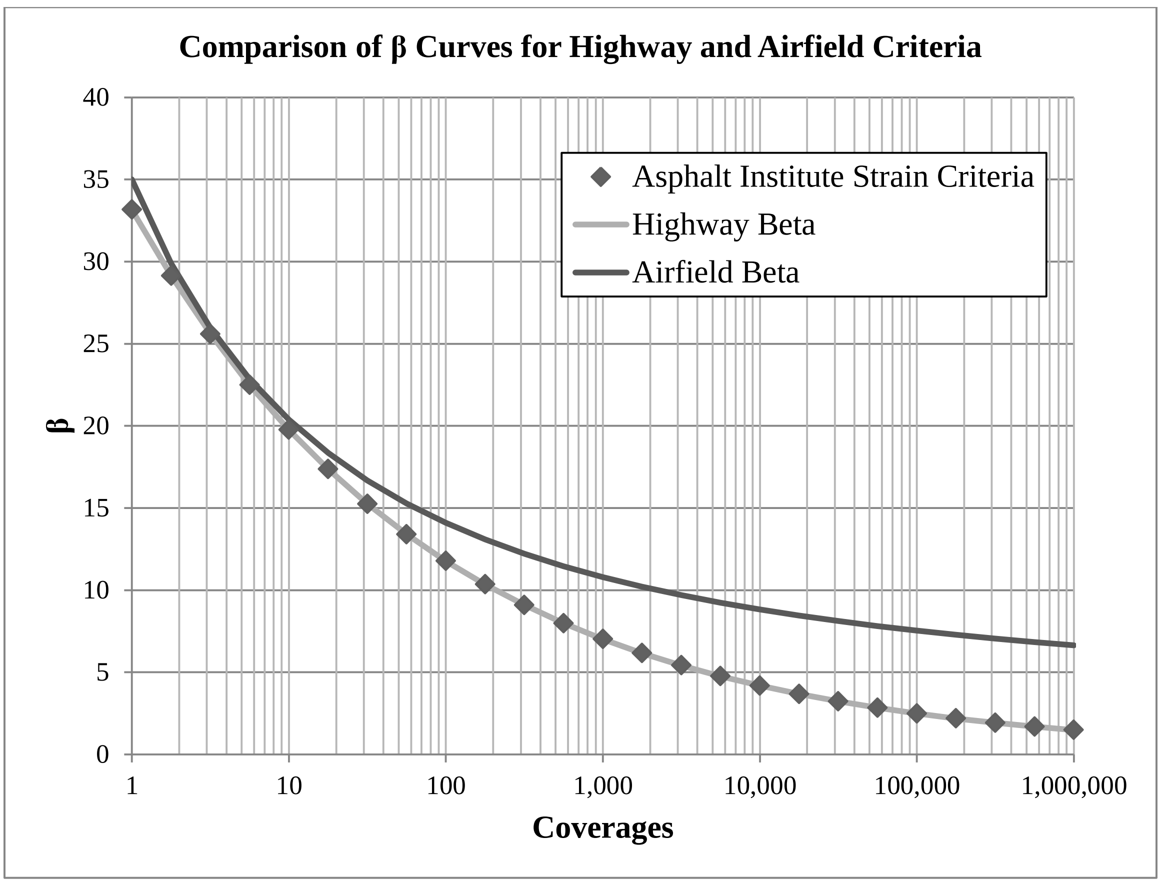

Figure 26. Comparison of β Curves for Highway and Airfield Criteria

Using the new highway β model, the concentration factor was optimized to fit the highway test sections. After seeing success with the two-layer model formulation for the airfield criteria, it appeared this formulation was reasonable as a starting point for this optimization process. For the same reasons seen with the airfield formulation, normal optimization tools could not be used due to the additional iterative dimension that occurred as a result of using the predicted thickness to calculate the concentration factor. As such, the optimization was conducted by hand. After approximately 50 trials, a model was created that produced the lowest sum of the squared error (SSE) score; this model is presented below as Equations 31 and 32 and shown graphically as Figure 27:

$$n_{SB} = 1.5\left(\frac{CBR_{SB}}{20}\right)^{\frac{0.5r}{t_{ac+base}}} + 0.25\ln(t_{ac+base}) + 0.1\sqrt{\frac{t_{ac}}{3}} + 0.25 \tag{31}$$

$$n_{SG} = \left(\frac{CBR_{SG}}{6}\right)^{\frac{1.75r}{t_{total}}} - 0.2(t_{ac+base})^{\frac{0.5}{n_{SB}}} - 0.05\ln(t_{total}) + 0.4 \tag{32}$$

where,

t_{total} = Thickness of Above Subgrade
$t_{ac+base}$ = Thickness of Above the Subbase (Wearing and Base Courses)
t_{ac} = Thickness of the Wearing Course
r = Contact Radius
CBR_{SB} = California Bearing Ratio of the Subbase Material
CBR_{SG} = California Bearing Ratio of the Subgrade Material
n_{SB} = Subbase Stress Concentration Factor
n_{SG} = Subgrade Stress Concentration Factor

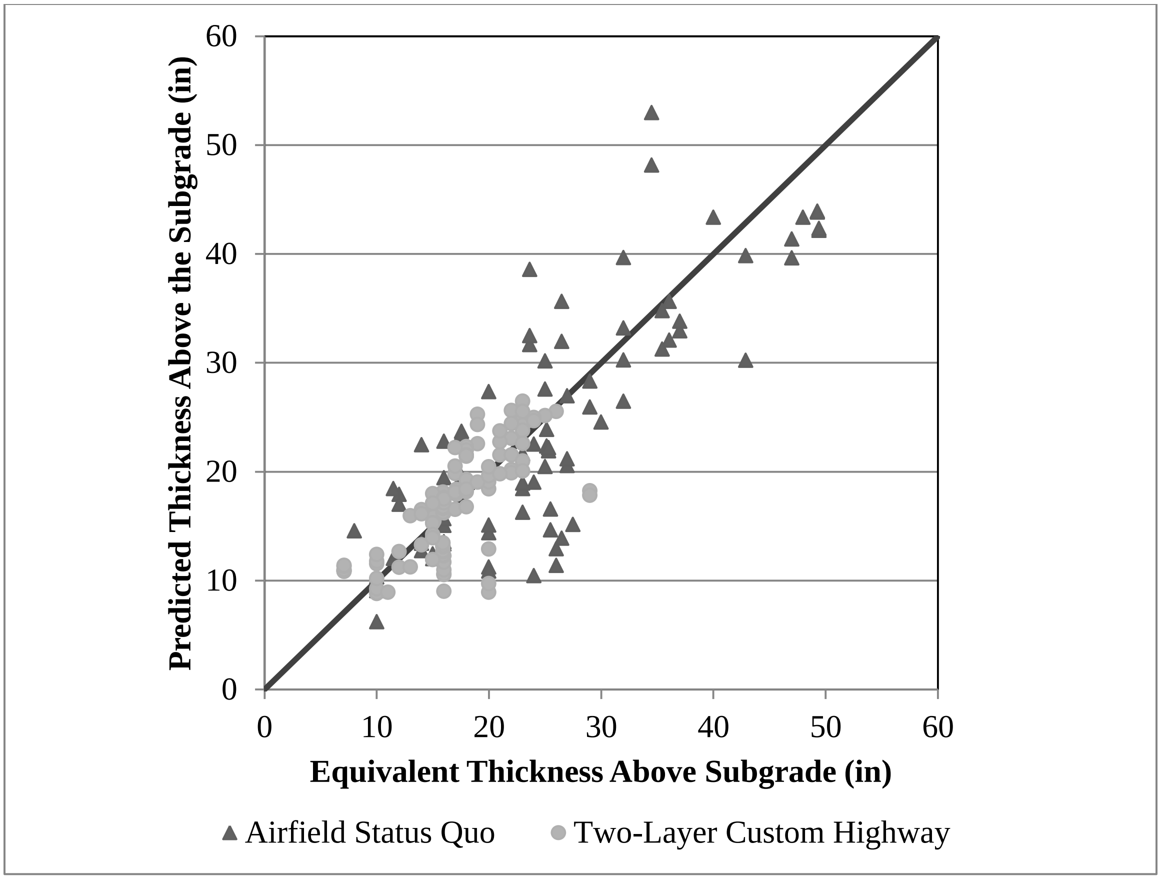

Figure 27. Comparison of Equivalent and Predicted Thicknesses for Highway Test Sections using the Modified Two-Layer Concentration Factor Model with Subgrade Strain Criteria Derived β Formulation (n = 82 – Airfield; n = 123 – Highway)

Analyzing the new highway design criteria for the CBR-Beta method in terms of APE resulted in decreases of 35 and 44 percent in MAPE and MdAPE scores, respectively, relative to the status quo airfield design criteria; this is shown in Figure 28. The MAPE and MdAPE scores for the new highway criteria were 0.15 and 0.10, respectively. By removing a few outlier data points, these scores would decrease even further.

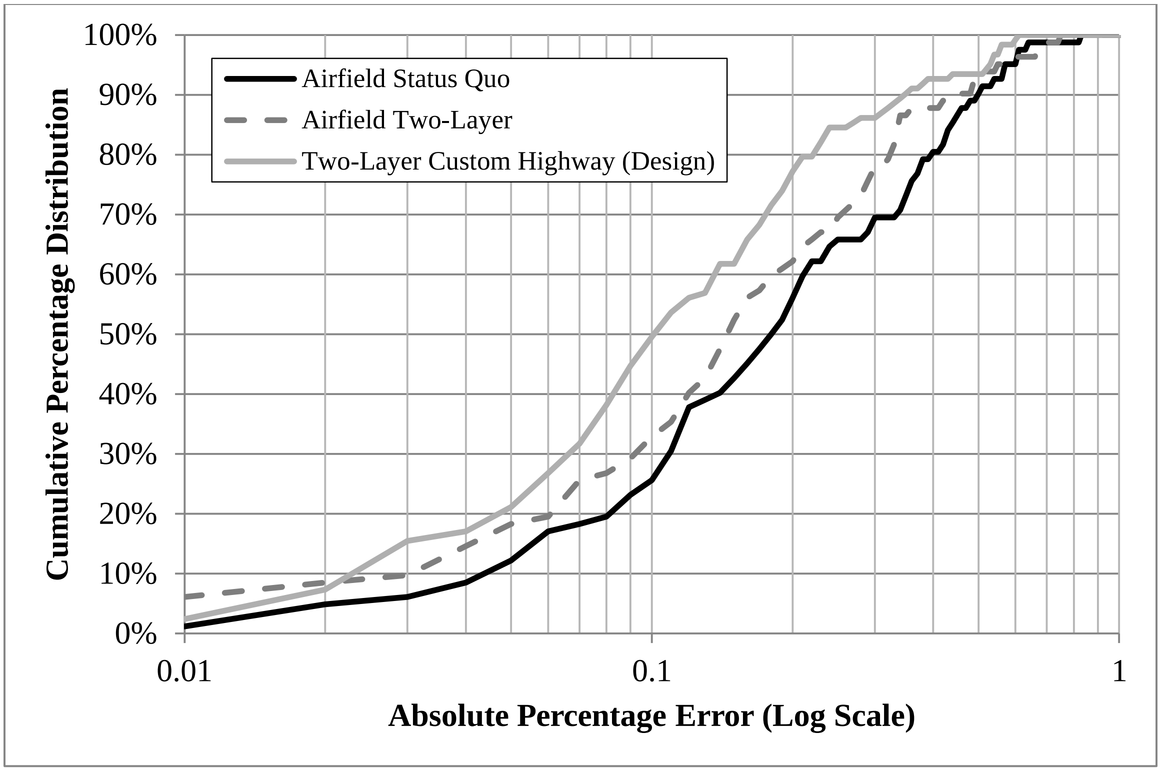

Figure 28. Comparison of the Cumulative Distributions of the Absolute Percentage Error Values for Highway and Airfield Pavements using CBR-Beta with Modified Highway Criteria (n = 82 – Airfield; n = 123 – Highway)

Summary and Conclusions

Due to different subgrade stress and strain criteria between airfield and highway pavements, the airfield criteria could not be directly applied to highway test sections with a reasonable degree of predictability. However, by utilizing the highway strain criteria to reformulate β for highway pavements, the performance of the highway test sections could be accurately predicted. Using the strain criteria from the Asphalt Institute, as it was empirically derived from evaluating highway test sections, there is an inherent characteristic of variability infused into the β formulation. As such, it is recommended that the results from this paper be verified using measured subgrade stress values prior to implementation for highway design and evaluation. Unfortunately, a relationship between the airfield and highway test sections could not be determined; however, through modification and optimization, the CBR-Beta method was extended to highway pavements. As a general note, with the proposed highway model and known layer thicknesses for the test sections, as would be the case for a pavement evaluation, the MAPE and MdAPE scores for the model decrease to 0.133 and 0.088, respectively. This improvement is shown in Figure 29.

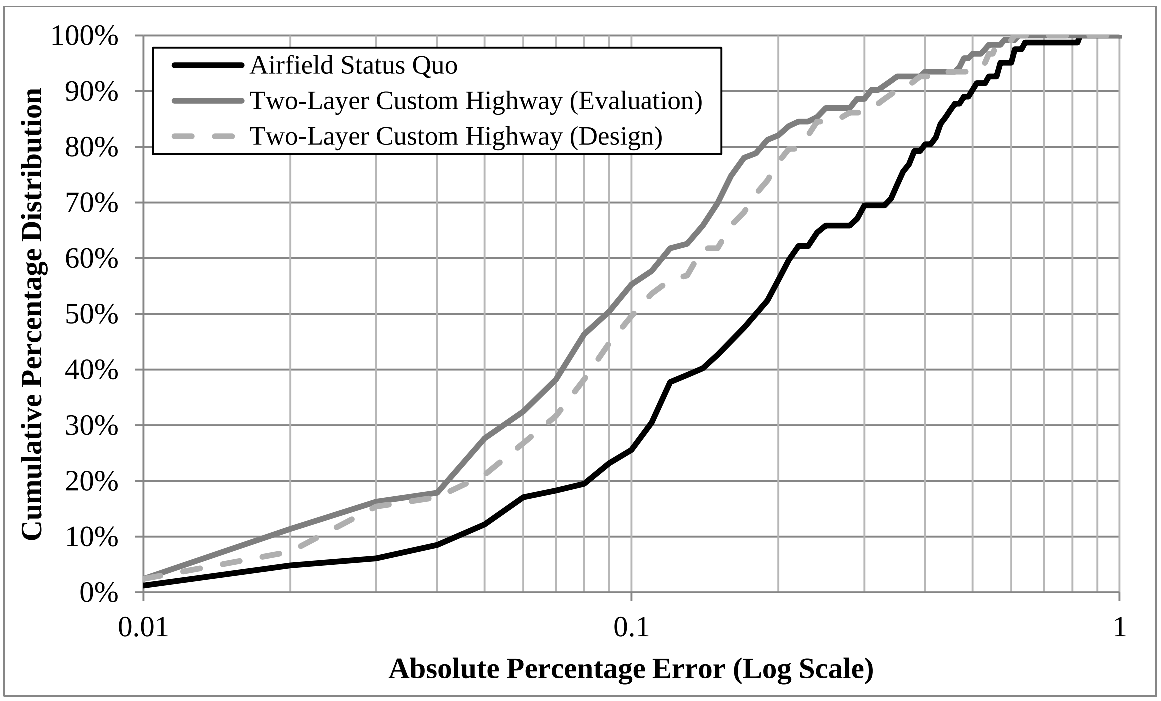

Figure 29. Comparison of Equivalent and Predicted Thicknesses for Highway Test Sections using the Customized Design Criteria with Known Layer Thicknesses (n = 123)

Disclaimer

The views expressed in this white paper are those of the author and do not reflect the official policy or position of the United States Air Force, Department of Defense, or the United States government.

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VII. Conclusions and Recommendations

This chapter summarizes the results presented in the previous three scholarly articles, the white paper, and their supporting appendices. While each of the articles and the white paper contain their own conclusions, this chapter combines the results into a consolidated recommendation regarding flexible airfield pavement design and evaluation. This chapter also addresses the completion status of each of the research objectives and concludes with a synopsis of the significance of the research, as well as recommendations for future research.

Conclusions of Research

The overall research objective was to the answer the question: how can the Air Force's current equivalency factors be adjusted to more accurately represent the actual structural capacity of stabilized layers by either developing new factors or adopting factors in-use by other organizations? Ultimately, the initial question expanded to include an analysis of the current concentration factor model used by the California Bearing Ratio-Beta (CBR-Beta) design method to estimate the vertical stress on the subgrade. This expansion grew out of the initial investigation of the stress distributions, in which it was identified that the stress distributions of non-stabilized pavements were not accurately modeled by the current concentration factor model. Additionally, at the request of the research sponsor, a brief investigation was conducted to analyze the potential of adding the more abundant highway testing data to the airfield testing data to increase the sample size used for the derivation of the empirical variables. Included below is a brief synopsis of the status of each research objective based upon the research.

Research Objective #1

> Assess the accuracy of the Air Force's current equivalency factors using test section data from previous full-scale, accelerated pavement tests to evaluate the ability of the factors to accurately predict the structural capacity of stabilized soils in flexible pavement systems.

As discussed in detail in Chapter III, the recommended equivalency factors from this study represented a significant change from the set of equivalency factors used currently. Looking at the base course equivalency factors in Table 16, the analysis from this research contradicted the current factor's assertion that cement-stabilized offered more thickness reduction than the asphalt-stabilized base course. This change aligns the equivalency factors with the Federal Aviation Administration (FAA) equivalency factors in this regard, although the magnitudes vary slightly due to variations in the equivalent thickness calculation and the FAA's use of a different design method. For the base course factors, enough data existed to draw reliable conclusions; however, the subbase factors were missing significant amounts of data to make conclusions with a high degree of confidence. Therefore, the values were forwarded to the Air Force Civil Engineer Center (AFCEC), with the caveat that further investigation was necessary to validate the recommendations.

Table 16. Comparison of the Current U.S. Army and Air Force Equivalency Factors to Recommended Equivalency Factors

		Equivalency Factors	
Course	**Stabilizer**	**Current**	**Modified**
Base	Asphalt-stabilized GW, GP, GM, GC	1.00	1.25
	Cement-stabilized GW, GP, SW, SP	1.15	1.00
Subbase	Asphalt-stabilized SW, SP, SM, SC	1.50	1.50
	Cement-stabilized ML, MH, CL, CH	1.70	2.20
	Cement-stabilized SC, SM	1.50	1.25
	Lime Stabilized ML, MH, CL, CH	1.00	1.10
	Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH	1.30	1.30
	Crushed Aggregate (P-209)	1.00	1.35

Research Objective #2

> Assess the accuracy of equivalency factors or methods used by other organizations to predict the structural capacity of stabilized soils using the previously mentioned test sections, and compare the predicative ability of these factors relative to the Air Force's equivalency factors.

As mentioned in Chapter III, each design method relies on different formulations for equivalent thickness and makes different assumptions in its respective design process. As such, a direct comparison of the factors was inappropriate; however, the factors were analyzed by looking at the trends within each set of equivalency factors from the various agencies. For example, the base course equivalency factors for this study ranked the asphalt-stabilized material higher than the cement-stabilized material. With the exception of the FAA, the other three sets of equivalency factors suggested the asphalt-stabilized base course was not significantly stronger than the cement-stabilized base course. Furthermore, the FAA suggests the improvement with asphalt-stabilized base is approximately 33 percent more than with cement-stabilized; the recommended factors for this study suggested this increase slightly lower at 25 percent. For the base courses, the recommended factors from this study tend to agree with the factors from the FAA. As a general note, the FAA specifies its equivalency factors as a range of values as a function of the modulus value; the values used in this study reflect an mean value.

To summarize, no other set of equivalency factors can be directly applied to the CBR-Beta method with a high degree of predictability for the reasons mentioned above. As such, the methodology utilized in this study was the most appropriate method, based on analyzing the failure coverages of test section data, to derive the equivalency factors as it customized the factors to the method and its assumptions. To refine the recommendations from this research, an analysis of the stresses in stabilized pavements under load could improve and validate the recommendations.

Research Objective #3

> Compare the life-cycle costs of the various flexible pavement design methodologies using standard conventional and stabilized pavement designs.

Based on the analysis, it was shown that the CBR-Alpha model produced thicker pavements on average than the other design methods. This conservative approach led to the CBR-Alpha method overdesigning a significant number of pavements, as shown by the comparison between the design methods based on predicting test section equivalent thicknesses. When a pavement is over-designed, it leads to a longer service life, albeit unintended. When using the CBR-Alpha model as the status quo, the other design models with higher predictability produced thinner pavements; therefore, these models theoretically supported fewer coverages. Comparing all of the design methods in terms of cost-per-pass, the CBR-Alpha method routinely produced the lowest average cost-per-pass; whereas the CBR-Beta method routinely produced higher average costs. CBR-Beta is more predictive than CBR-Alpha; however, the reduction in pavement thickness led to lower initial construction costs but higher operating costs over the reduced service life. In summary, the CBR-Beta did not result in pavements with a lower initial construction cost significant enough to outweigh the additional passes attributed to the conservative approach of the CBR-Alpha method.

By selecting a design method based solely on initial cost, predicted thickness, or accuracy in relation to failure passes of experimental data, decision-makers often overlook the long-term implications of such a decision. For example, the modified CBR-Beta model produced the thinnest pavements at the lowest cost and appears to offer the highest predictability with regard to historical experimental data; however, when the life-cycle of the pavement is considered, the modified CBR-Beta method results in the shortest service life of all the methods. Therefore, these pavements would potentially require additional full-depth repairs more frequently.

Research Objective #4

> Determine the cost of utilizing stabilized soils in lieu of hauling conventional materials.

From the research conducted in Chapter IV, there was no definitive conclusion in terms of whether a stabilized base course cost more than a conventional base. Ultimately, it depended on the method being used to design the pavement system. As shown in Figure 30, both the CBR-Alpha and CBR-Beta methods, with the modified equivalency factors for cement-stabilized base, resulted in higher initial construction costs relative to the cost of conventional construction for the respective method. The disparity in percentage difference between the modified and current equivalency factors for both the CBR-Alpha and CBR-Beta methods proved to be significant to this research objective, as the shift in equivalency factor from 1.15 to 1.00 resulted in cost increases of at least 129 percent over the course of the 81 trials.

Further analyzing Figure 30, the disparity in cost differences between LEDFAA and FAARFIELD was the result of minimum thickness criteria. LEDFAA, as analyzed in this study using criteria from AC 150-5320-6D, incorporated minimum thickness criteria that aligned closely with the current the USACE's current criteria. Conversely, FAARFIELD, with the criteria from AC 150-5320-6E, utilizes more relaxed minimum thickness criteria. This seemingly minor difference accounted for a 33 percent difference in costs over the 81 trials.

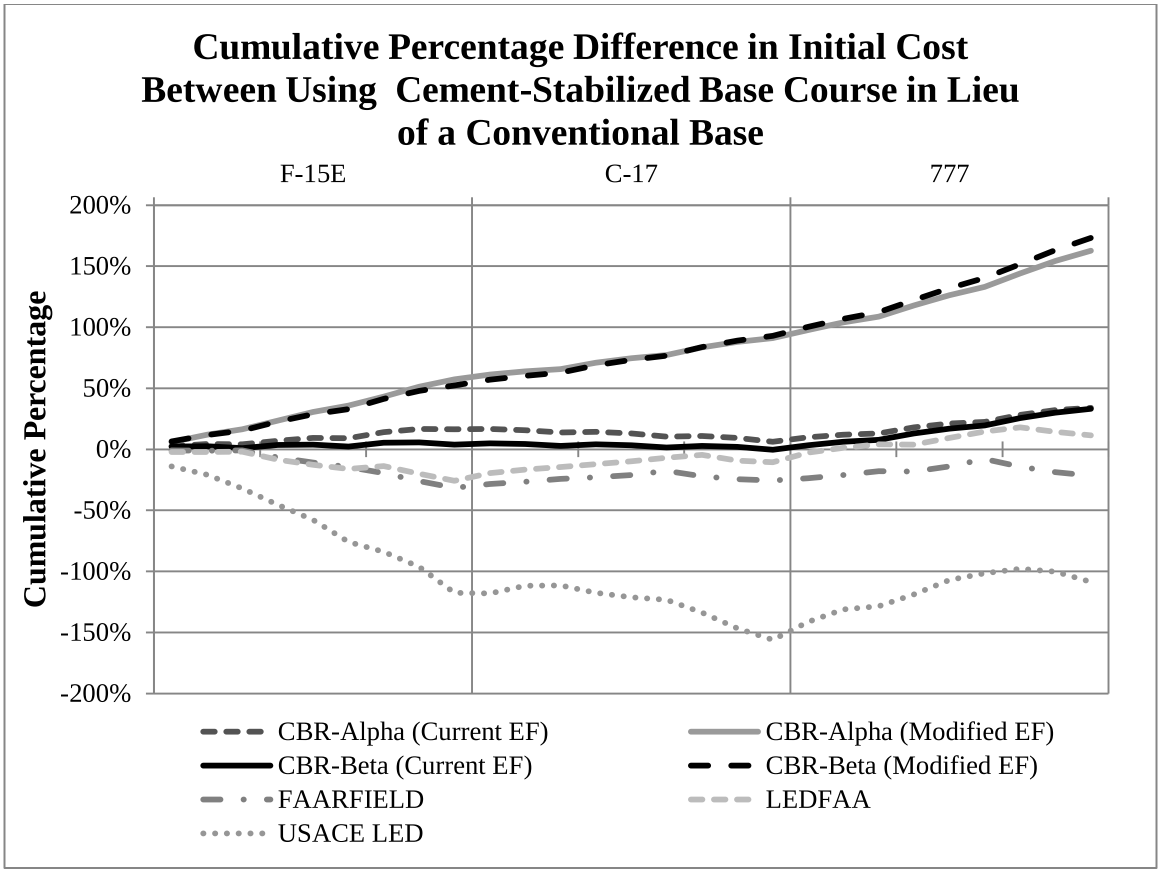


Figure 30. Cumulative Percentage Difference in Initial Cost Between Using Cement-Stabilized Base Course in Lieu of a Conventional Base (n = 27)

Similar to the commentary for cement-stabilized base courses, the asphalt-stabilized saw similar trends; however, the modified equivalency factors resulted in asphalt-stabilized base courses resulting in lower initial construction costs relative to conventional pavements. As shown in Figure 31, the shift in the equivalency factor from 1.00 to 1.25 resulted in a decrease of approximately 200 percent to the point where asphalt-stabilized pavements are cost-effective without the need for hauling. With the current equivalency factor of 1.00, the CBR-Alpha and CBR-Beta methods resulted in almost a 150 percent increase in cost over the use of conventional base courses. To offset this cost, conventional materials would need to be hauled over 100 miles to make asphalt-stabilized pavements cost effective.

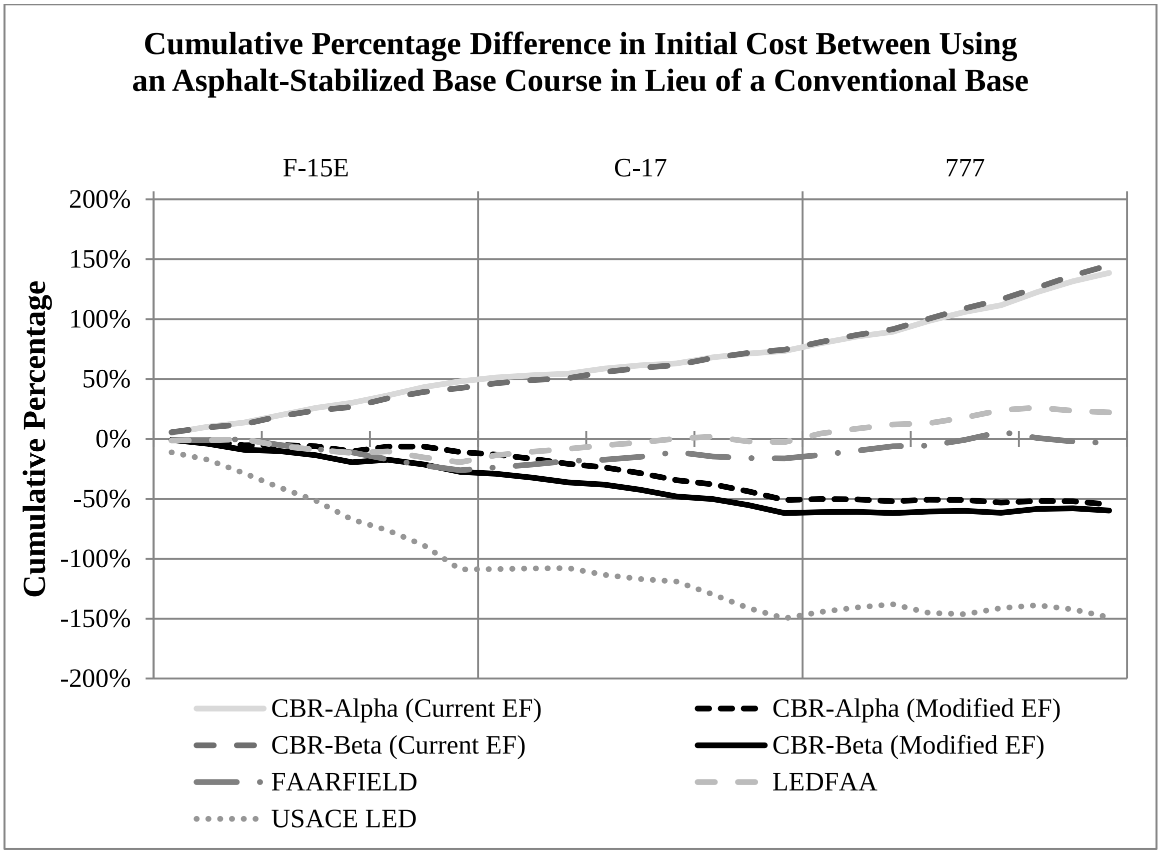


Figure 31. Cumulative Percentage Difference in Initial Cost Between Using an Asphalt-Stabilized Base Course in Lieu of a Conventional Base (n = 27)

Research Objective #5

> Validate the Air Force's use of the Boussinesq-Frohlich Model for pavement systems with stabilized layers.

This research objective remains unanswered in its entirety. Significant progress was accomplished during this research effort; however, more information is ultimately needed to fully address the issue. From this analysis, it appeared that a two-layer concentration factor model was better than a single-layer model at predicting both equivalent thickness above the subgrade and vertical stress on the subgrade. This improvement in statistical measures implied that the two-layer model more accurately characterized the stiffness of the wearing and base courses. The two-layer model, although admittedly requiring further refinement, appeared to show that the stress on the subgrade is dependent on the stiffness of the layers above it; in this situation, the formulation of the subgrade concentration factor included the subbase concentration factor and the thicknesses of the wearing and base course layers. As such, the assumption of homogeneity throughout the depth of the pavement system when calculating the vertical stresses appeared to be overly simplistic relative to the two-layer formulation in that the assumption of homogeneity does not account for the load distributing properties of the layers above the subgrade.

In Chapter V, it was discussed that there appeared to be a disparity between the allowable stress criteria and the measured stresses. This disparity caused the two-layer model, formulated using coverage-derived concentration factors, to result in high predictability with respect to predicting equivalent thickness, yet result in much lower correlations between predicted and measured stress. By shifting to stress derived concentration factors, the model, to include the status quo formulation, produced high predictability in predicting vertical stress, yet could not accurately predict equivalent stress. Breaking this problem down further, it appeared that the allowable stress criteria, and more specifically β, were the primary cause for this disparity.

To further test this assertion, a two-layer model was optimized based on stress-derived concentration factor values. Shifting to this model, represented in Figure 32, resulted in a 216 percent reduction in the sum of the squared error (SSE) scores relative to the two-layer model derived using failure coverages. Similar statements could be made regarding the other statistical measures of fit, such as Pearson's correlation coefficient.

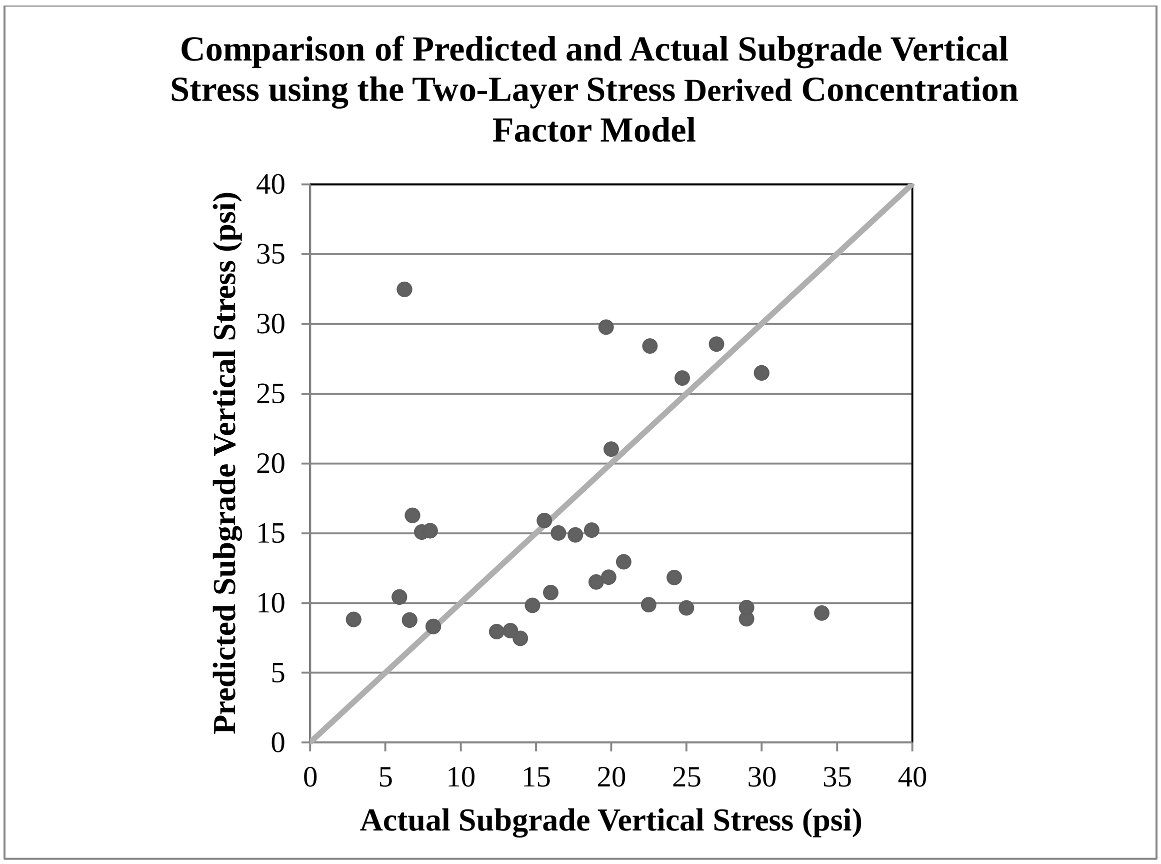


Figure 32. Comparison of Predicted and Actual Subgrade Vertical Stress using the Optimized Two-Layer Stress Derived Concentration Factor Model (n = 41)

When the stress derived two-layer model was allowed to iterate through the CBR-Beta process to predict the thickness required to protect the subgrade, the model could not accurately predict the equivalent thicknesses of the test sections it was attempting to replicate; this is shown in Figure 33. The stress derived predicted thicknesses resulted in a significantly higher error rate than the failure derived model; the stress derived model increased the SSE score for the model by 506 percent. This disparity between the two model derivations appeared to exist as a result of the allowable stress criteria, as this criteria was the only variable linking the two models. As documented in Chapter V, revising β had a definitive effect on aligning the stress and failure derived concentration factors.

Additional Research on Applicability of Highway Testing Data

Due to different subgrade stress and strain criteria between airfield and highway pavements, the airfield criteria could not be applied directly to highway test sections with a reasonable degree of predictability. By utilizing the highway strain criteria to reformulate β for highway pavements, the performance of the highway test sections could be accurately predicted. Using the strain criteria from the Asphalt Institute, as it was empirically derived from evaluating highway test sections, there is an inherent characteristic of variability infused into the β formulation. As such, it is recommended that the results from this paper be verified using measured subgrade stress values prior to implementation for highway design and evaluation. Unfortunately, a relationship between the airfield and highway test sections could not be found; however, through modification and optimization, the CBR-Beta method was successfully extended to highway pavements.

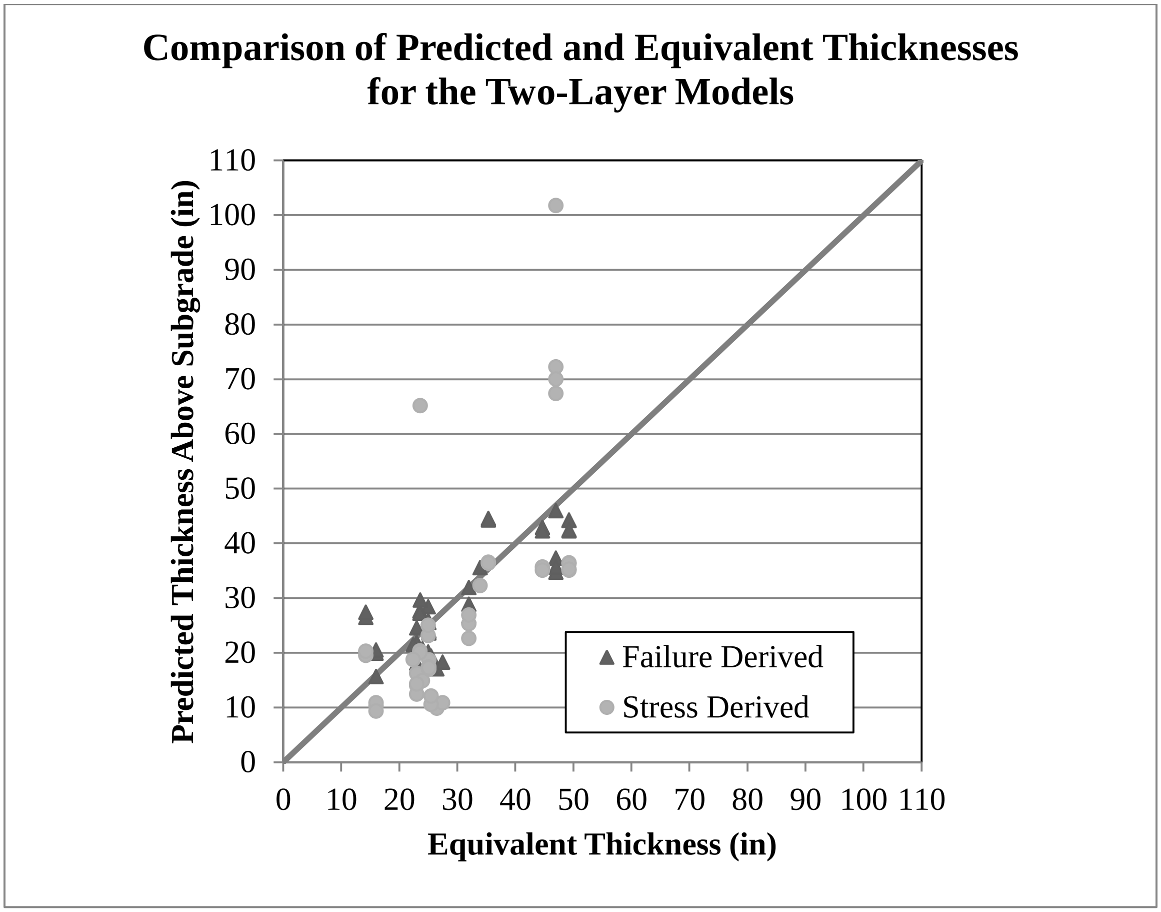


Figure 33. Comparison of Predicted and Equivalent Thicknesses for the Two-Layer Models Derived using Failure Coverages and Actual Vertical Stress (n = 41)

Significance of Research

The overall goal of the research effort was to establish more realistic equivalency factors for the design and evaluation of military airfields and to analyze the procedures for designing and evaluating stabilized soils. As discussed in this chapter within the research objectives section, this research effort analyzed several difference aspects pertaining to stabilized soils. With the cost analysis performed, decision makers can understand the true opportunity costs of utilizing stabilized soils in lieu of conventional soils. Furthermore, decision-makers can realize that by shifting from an overly conservative design method to a more predictive method, additional and unforeseen costs will exist, as the more predictive design method will not produce as many over-designed pavements, thus reducing the probability of pavements greatly exceeding their design life. Shifting focus to the equivalency factors and the concentration factor study, this research demonstrated that a two-layer concentration factor model combined with modified equivalency factors can reduce the median and mean APE scores (MdAPE and MAPE, respectively, in Table 17), as well as minimize the disparity in these scores between stabilized and non-stabilized pavements when analyzed as subgroups.

Table 17. Comparison of the Mean and Median Absolute Percentage Errors for the Status Quo Concentration Factor Model with the Current Equivalency Factors and the Two-Layer Failure Derived Model with Modified Equivalency Factors (n = 157)

	Status Quo with Current Equivalency Factors			**Two-Layer Failure Derived Model with Modified Equivalency Factors**		
	Non-Stabilized	**Stabilized**	**Δ**	**Non-Stabilized**	**Stabilized**	**Δ**
MAPE	0.231	0.251	0.200	0.179	0.187	0.008
MdAPE	0.178	0.189	0.011	0.145	0.153	0.008

Overall, this research improved upon the flexible pavement design and evaluation with stabilized soils using equivalency factors. Secondly, this research expanded the body of knowledge with respect to the concentration factor formulation for the CBR-Beta design method by demonstrating evidence that a two-layer concentration factor model can be used accurately for both pavement design and evaluation. These two contributions contribute to reducing the error rate for design and evaluation, thereby reducing the probability of early failure due to under-design and saving costs by reducing the probability of over-designed pavements.

Recommendations for Future Research

This research represented a small portion of a larger effort to review the U.S. Army Corps of Engineers' (USACE) and the AFCEC's pavement programs in its entirety. Starting with topics specifically identified in consultants' assessments of the programs, such as this topic, the USACE and the AFCEC are beginning to finalize its long-term research plan. To support this effort, the following potential topics might be of interest to further refine the pavement programs.

- Investigate the equivalent thickness formulation.
- Perform a meta-analysis to characterize the vertical stress above the subbase, particularly as it pertains to stabilized layers.
- Review the allowable stress criteria to align the measured stress with the allowable stress.
- Explore the possibility of using finite-element analysis for flexible pavements.
- Verify proposed highway design criteria using measured subgrade stress and strain values.

Appendix A. Supporting Documentation for Investigation of Current Equivalency Factors Usages by the Department of Defense and Various Outside Agencies

The general results included in this section are broken into categories based upon the equivalency factor analyzed. Included with each equivalency factor is a histogram of the actual data, a histogram of the simulated data, a cumulative distribution plot showing both the actual and simulated data, and an optimization curve. As previously mentioned, due to limited data availability, this study only looked at eight of the U.S. Air Force and Army's equivalency factors. The intermediate results from this study are presented in the subsequent paragraphs; the overall results are presented in Table **16**.

Base Course Equivalency Factors

Table 18. Modified Base Course Equivalency Factors

		Equivalency Factors	
Course	**Stabilizer**	**Current**	**Modified**
Base	Asphalt-stabilized GW, GP, GM, GC	1.00	1.25
	Cement-stabilized GW, GP, SW, SP	1.15	1.00

Asphalt-Stabilized GW, GP, GM, GC

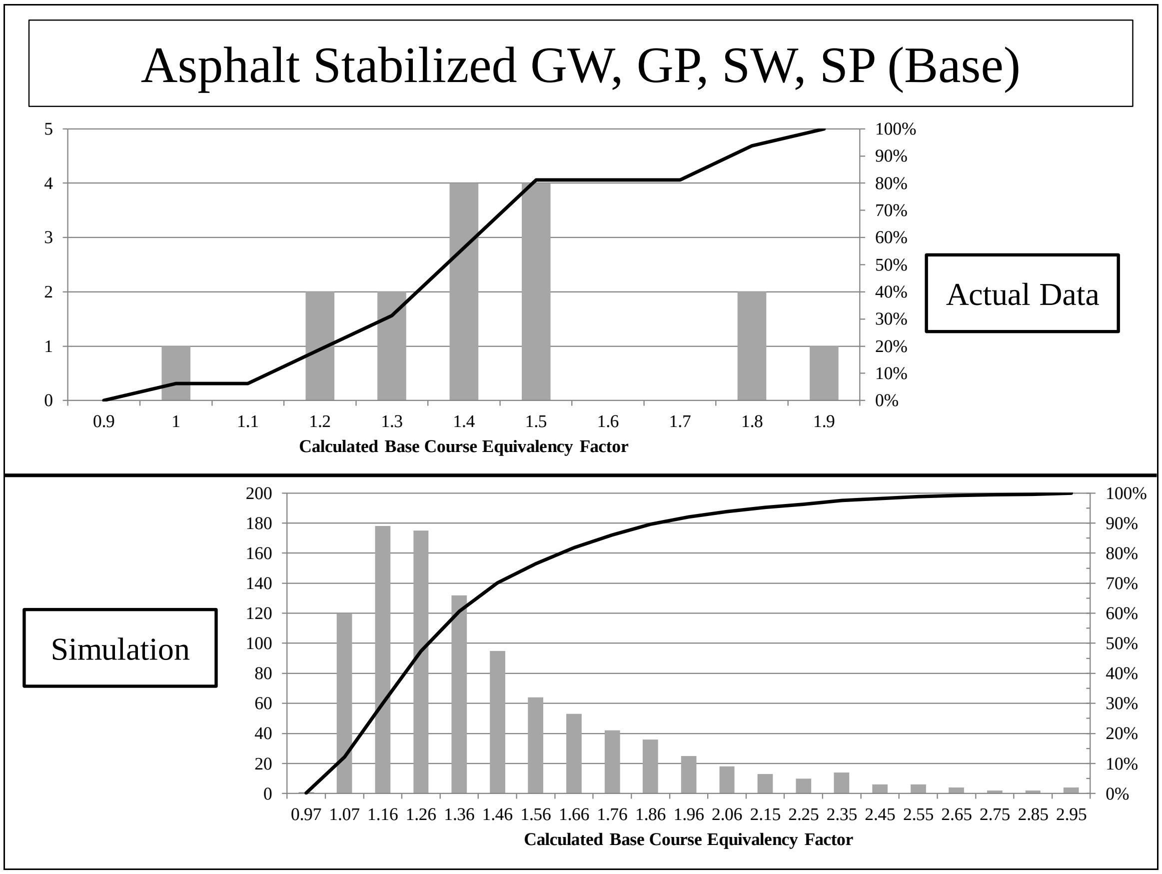


Figure 34. Comparison Between Actual and Simulated Data for Asphalt-Stabilized GW, GP, SW, SP (Base Course Equivalency Factor)

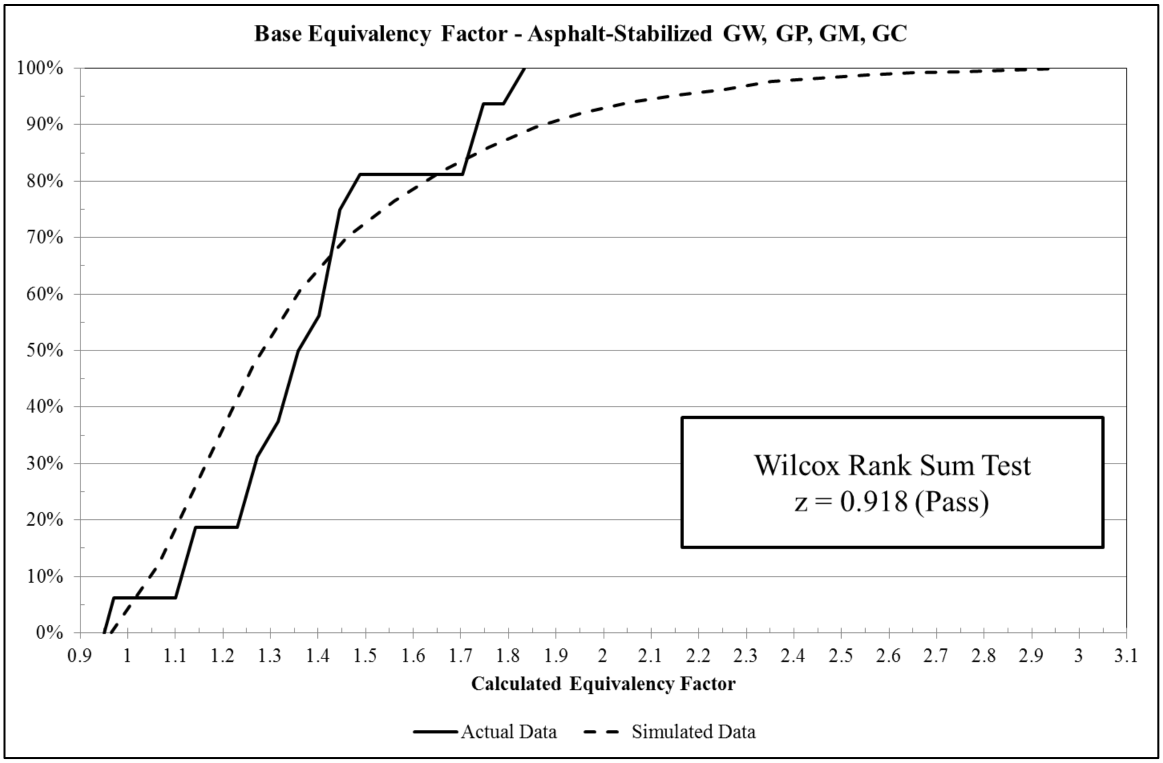


Figure 35. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Asphalt-Stabilized GW, GP, SW, SP (Base Course Equivalency Factor)

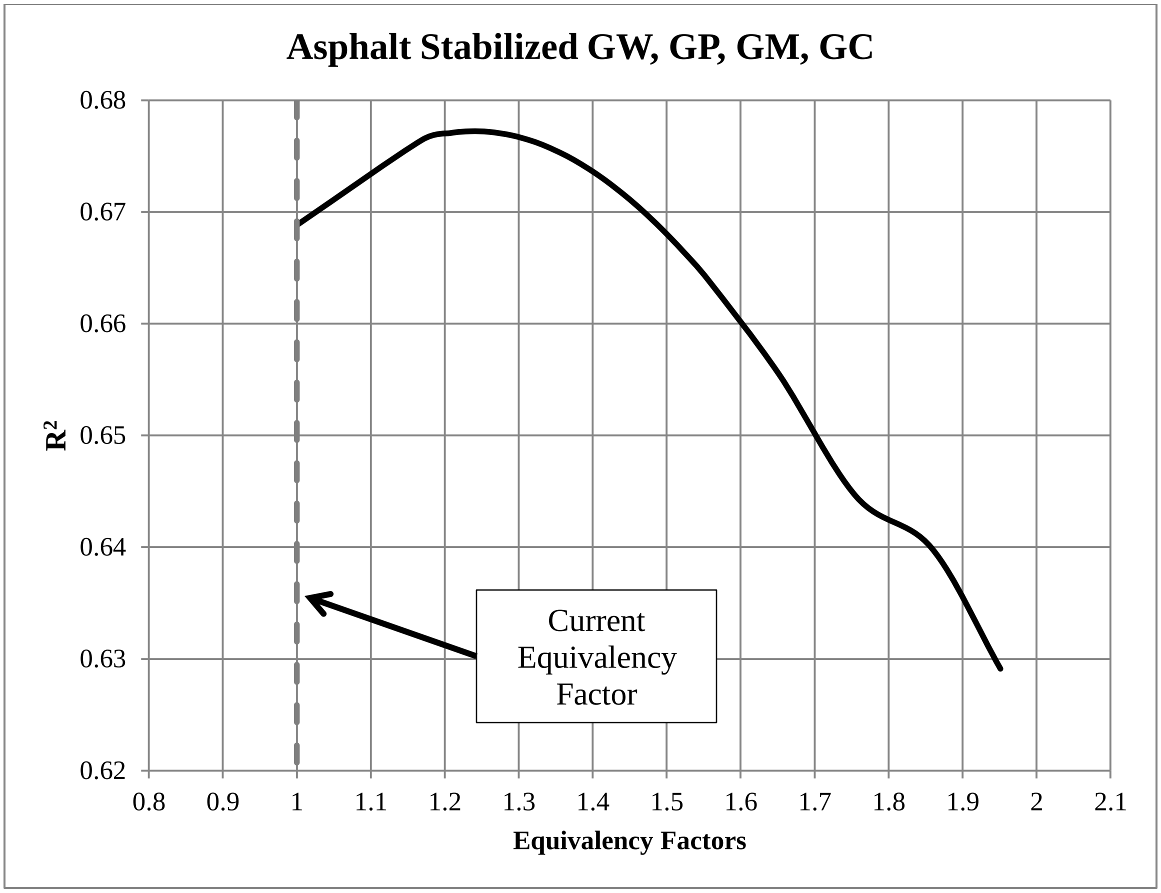


Figure 36. Optimization Curve for Asphalt-Stabilized GW, GP, GM, GC Base Course Equivalency Factor

Cement-Stabilized GW, GP, SW, SP

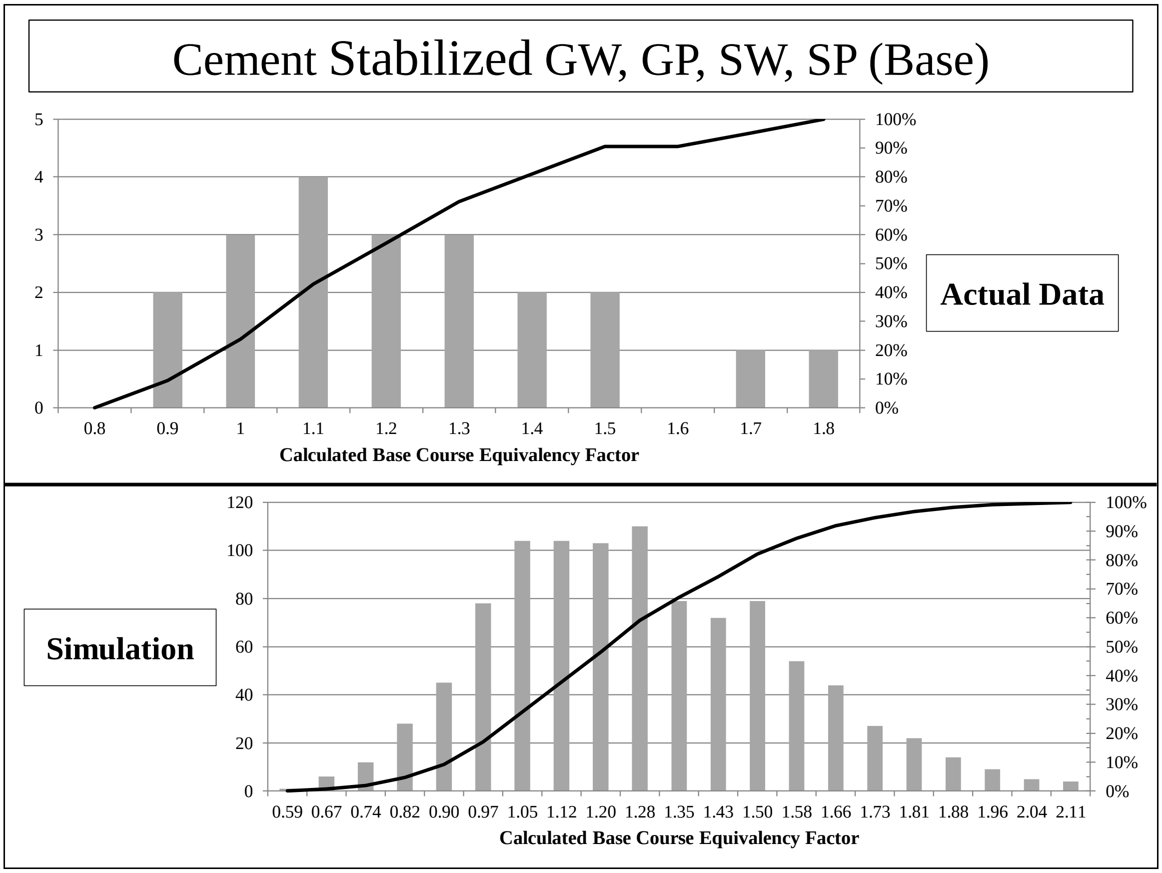


Figure 37. Comparison Between Actual and Simulated Data for Cement-Stabilized GW, GP, SW, SP (Base Course Equivalency Factor)

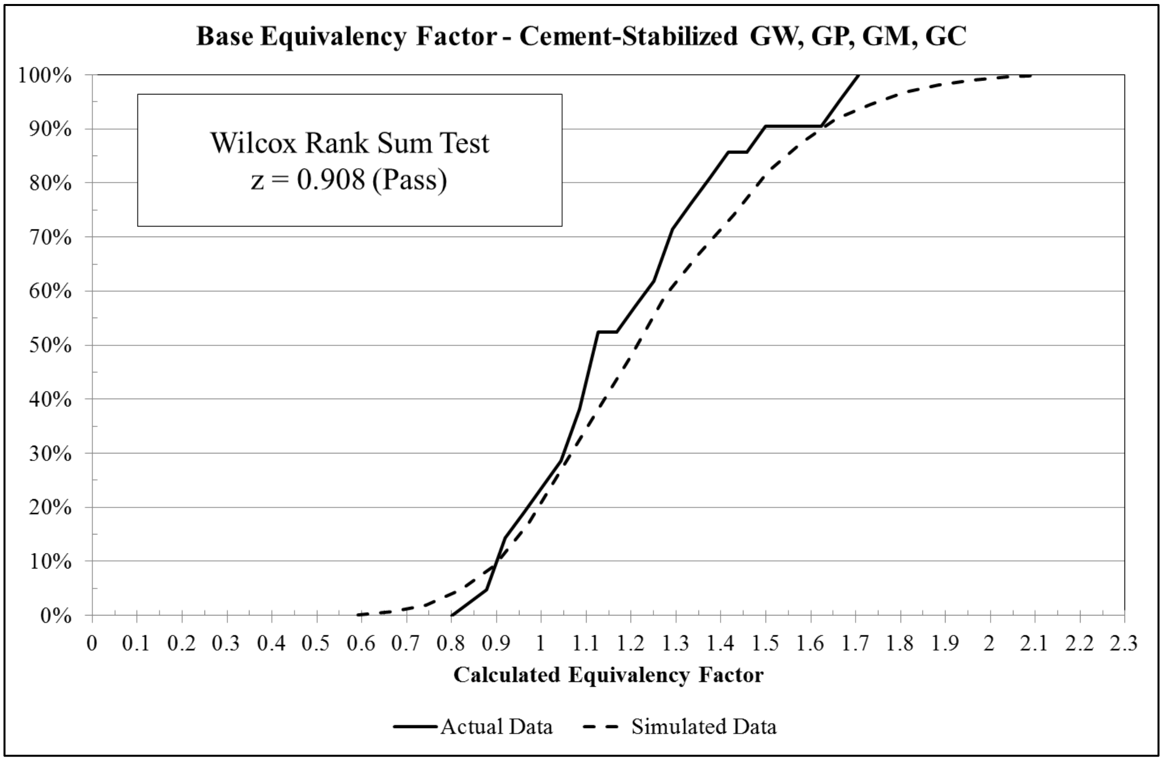


Figure 38. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Cement-Stabilized GW, GP, SW, SP (Base Course Equivalency Factor)

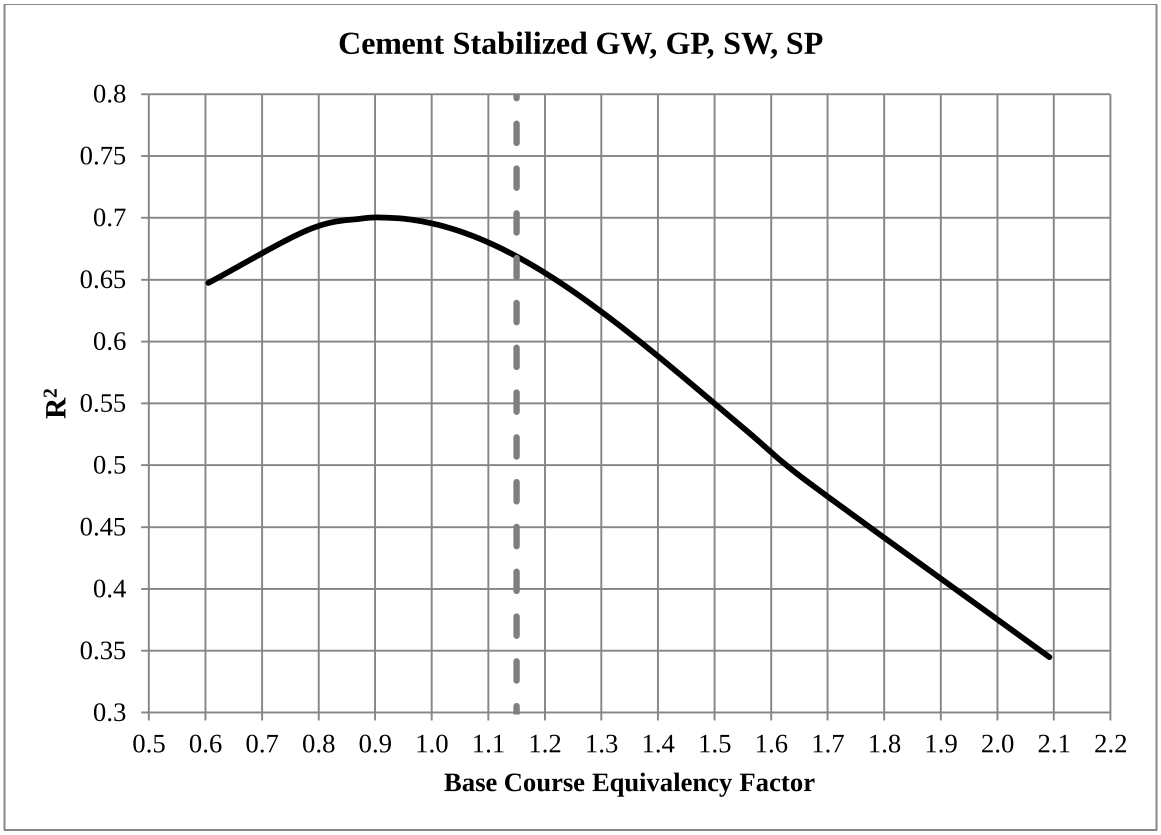


Figure 39. Optimization Curve for Cement-Stabilized GW, GP, GM, GC Base Course Equivalency Factor

Subbase Course Equivalency Factors

Table 19. Modified Subbase Course Equivalency Factors

		Equivalency Factors	
Course	**Stabilizer**	**Current**	**Modified**
Subbase	Asphalt-stabilized SW, SP, SM, SC	1.50	1.75
	Cement-stabilized ML, MH, CL, CH	1.70	2.20
	Cement-stabilized SC, SM	1.50	1.25
	Lime Stabilized ML, MH, CL, CH	1.00	1.10
	Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH	1.30	1.30
	Crushed Aggregate (P-209)	1.00	1.40

Asphalt-Stabilized GW, GP, GM, GC

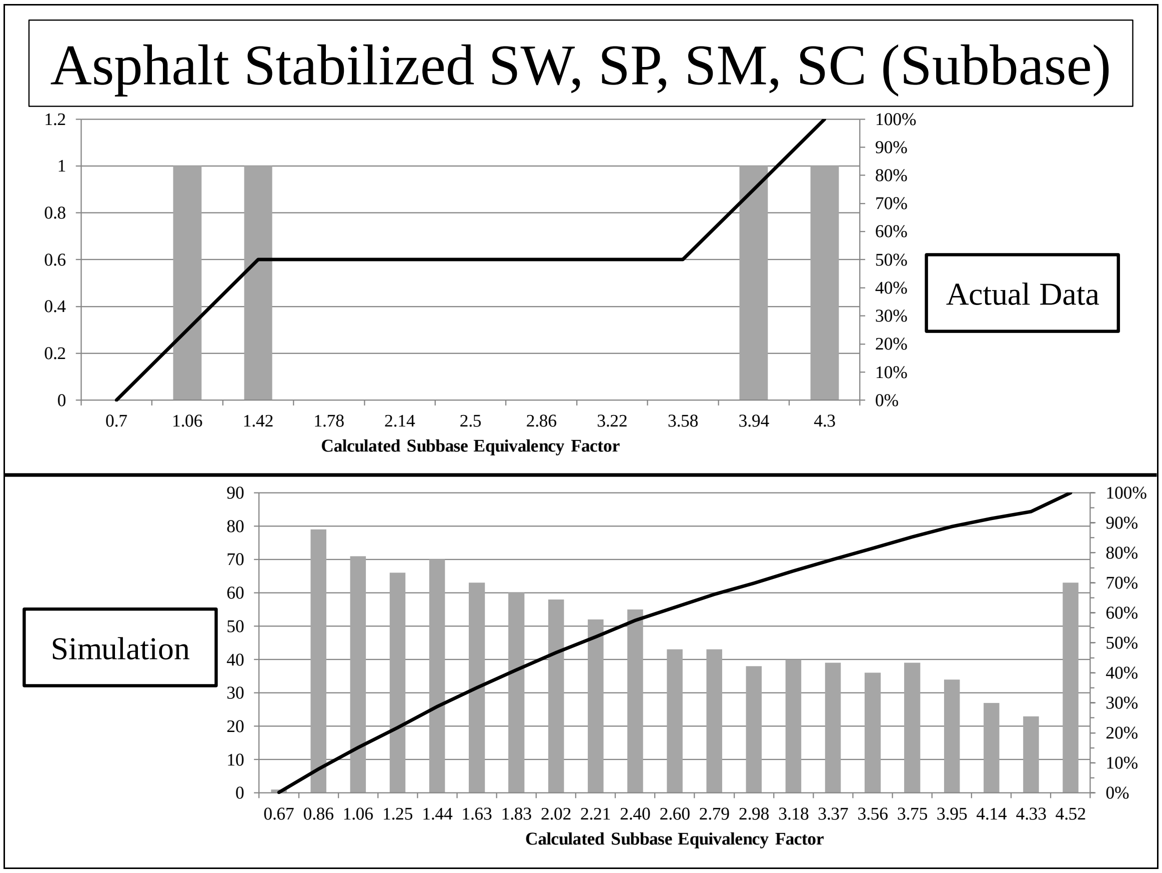


Figure 40. Comparison Between Actual and Simulated Data for Asphalt-Stabilized SW, SP, SW, SP (Subbase Course Equivalency Factor)

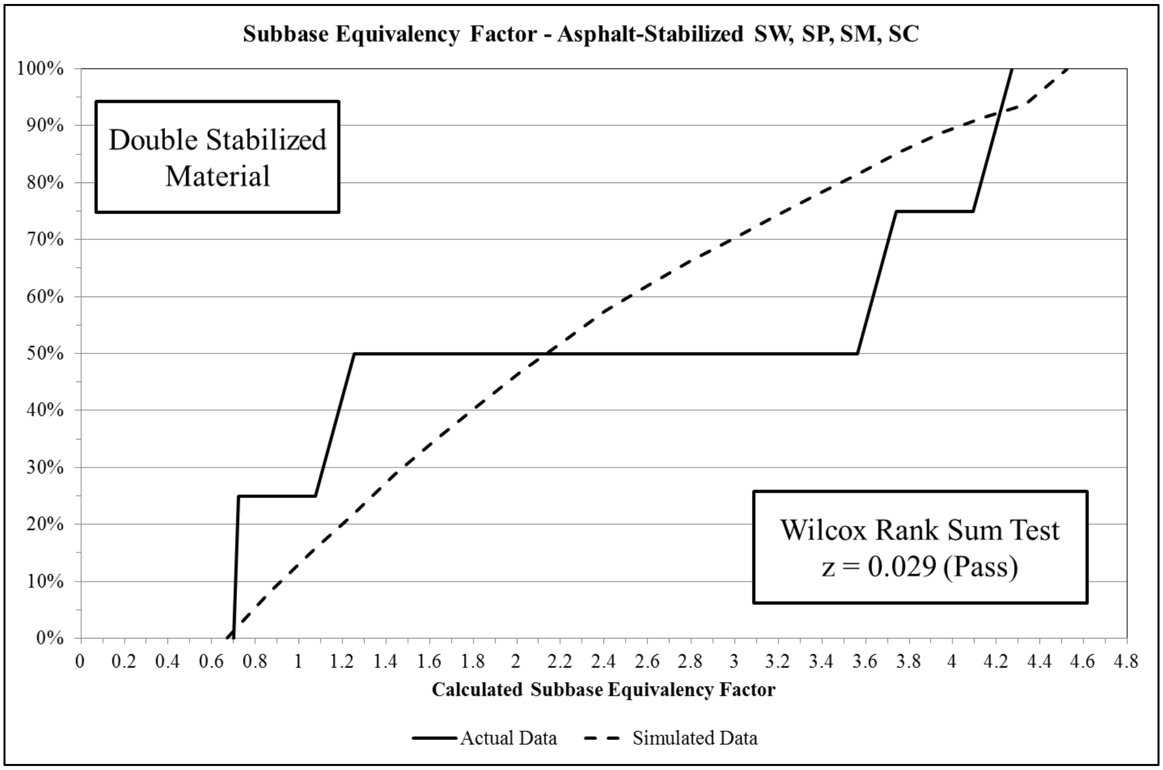


Figure 41. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Asphalt-Stabilized SW, SP, SW, SP (Subbase Course Equivalency Factor)

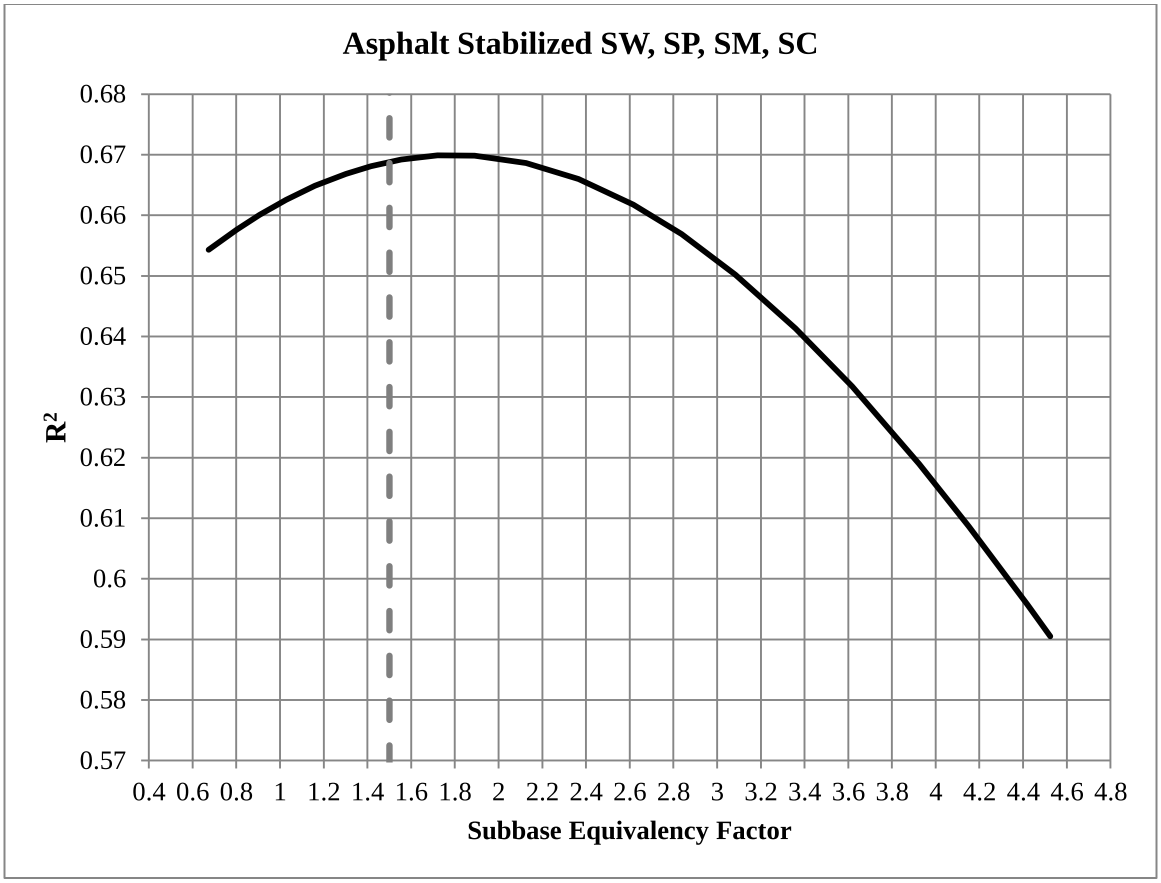


Figure 42. Optimization Curve for Asphalt-stabilized SW, SP, SM, SC Subbase Course Equivalency Factor

Cement-Stabilized ML, MH, CL,CH

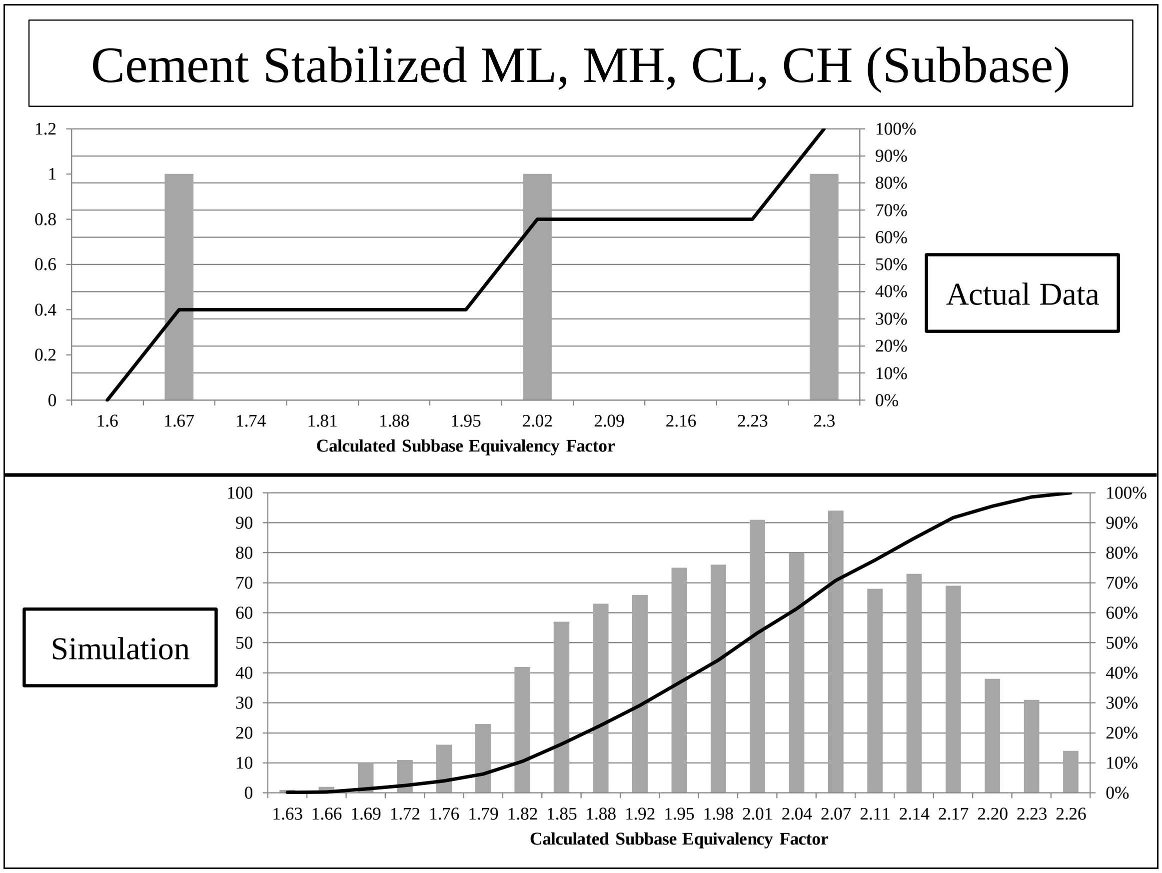


Figure 43. Comparison Between Actual and Simulated Data for Cement-Stabilized ML, MH, CL, CH (Subbase Course Equivalency Factor)

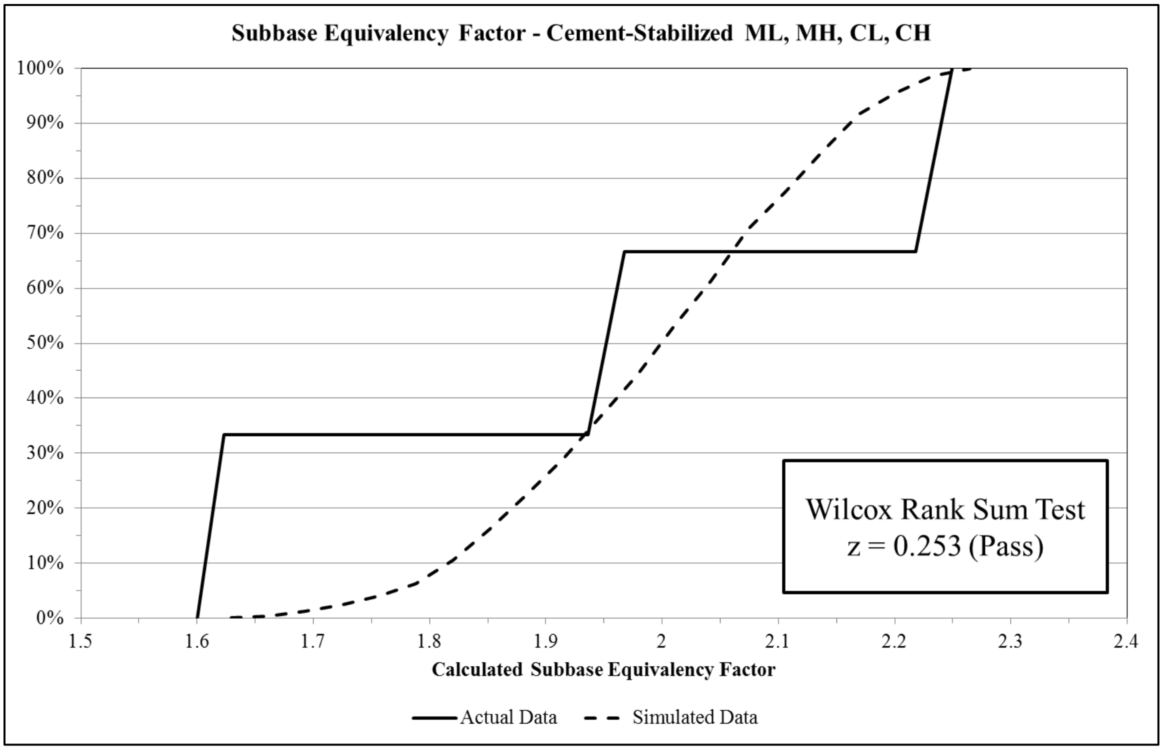


Figure 44. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Cement-Stabilized ML, MH, CL, CH (Subbase Course Equivalency Factor)

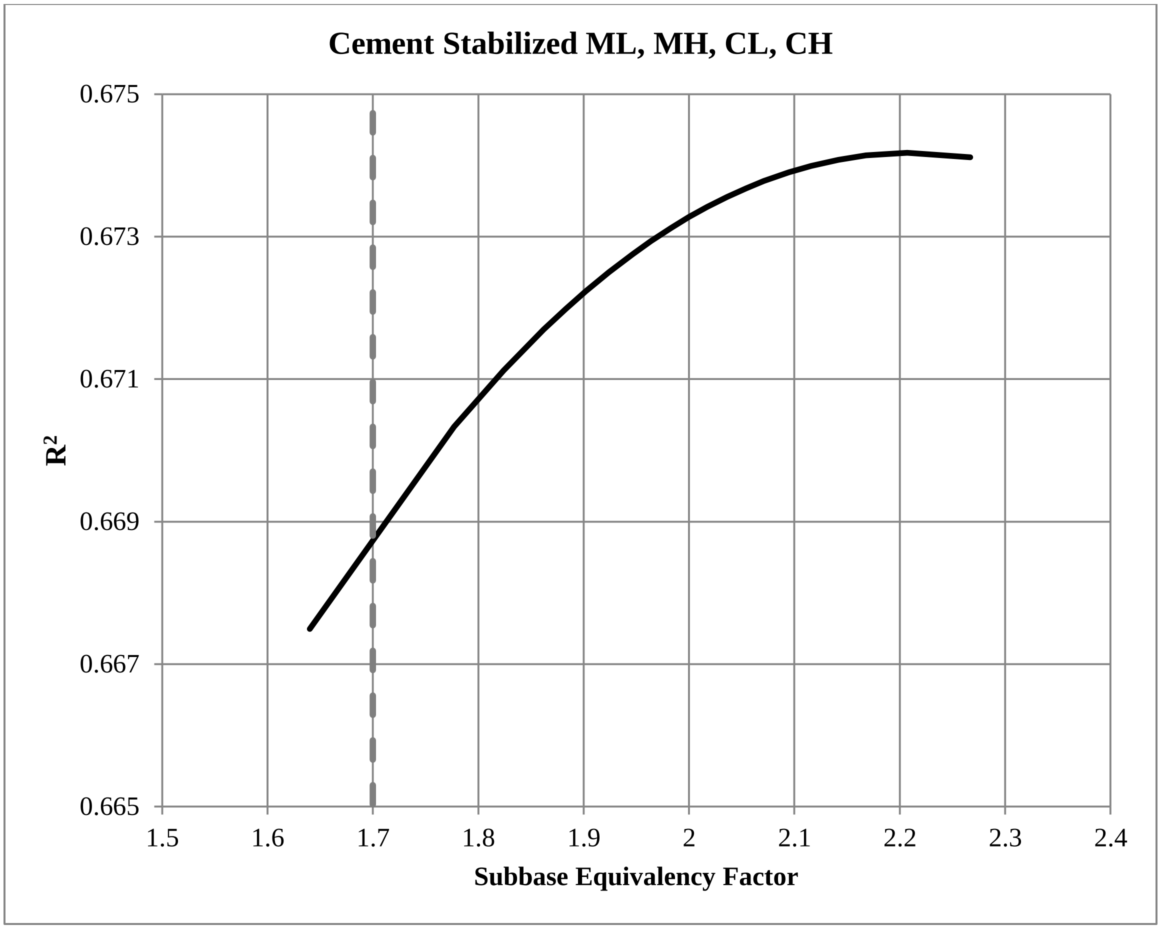


Figure 45. Optimization Curve for Cement-stabilized ML, MH, CL, CH Subbase Course Equivalency Factor

Cement-stabilized SC, SM

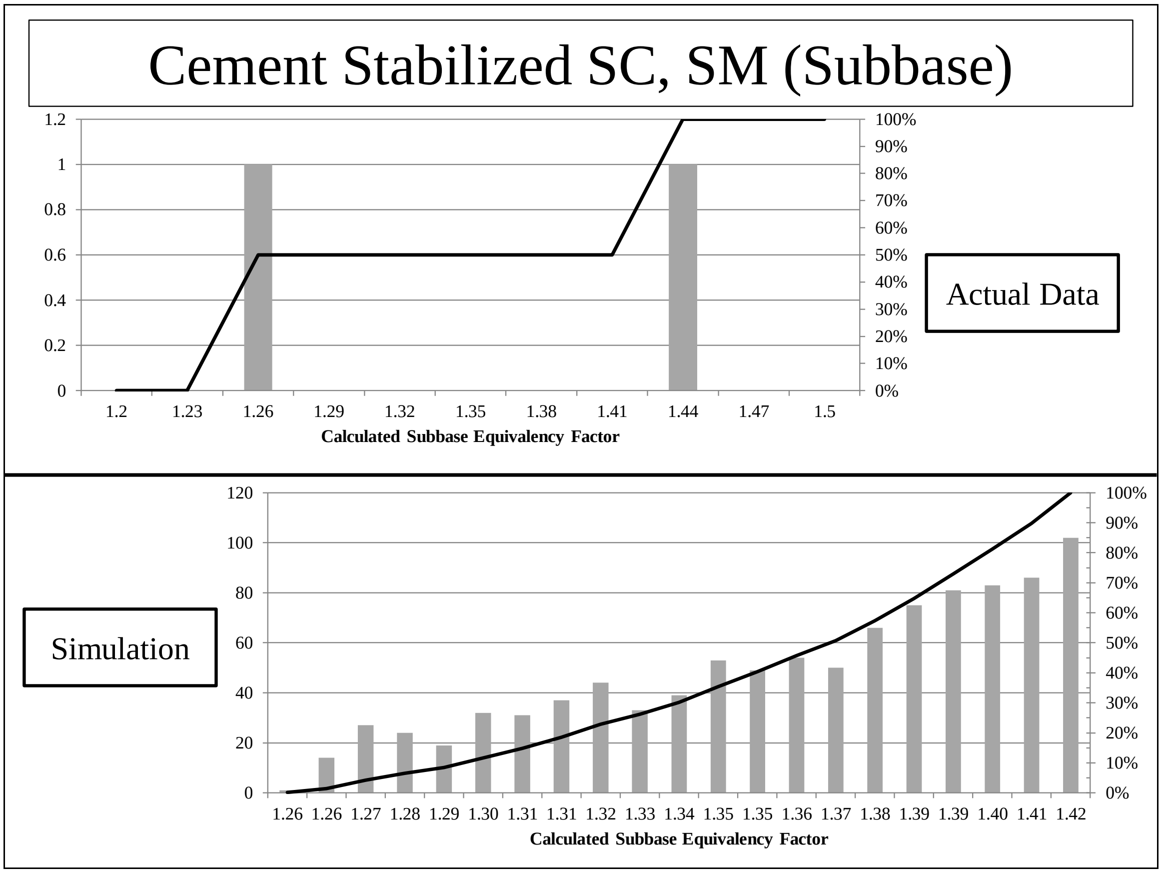


Figure 46. Comparison Between Actual and Simulated Data for Cement-Stabilized SC, SM (Subbase Course Equivalency Factor)

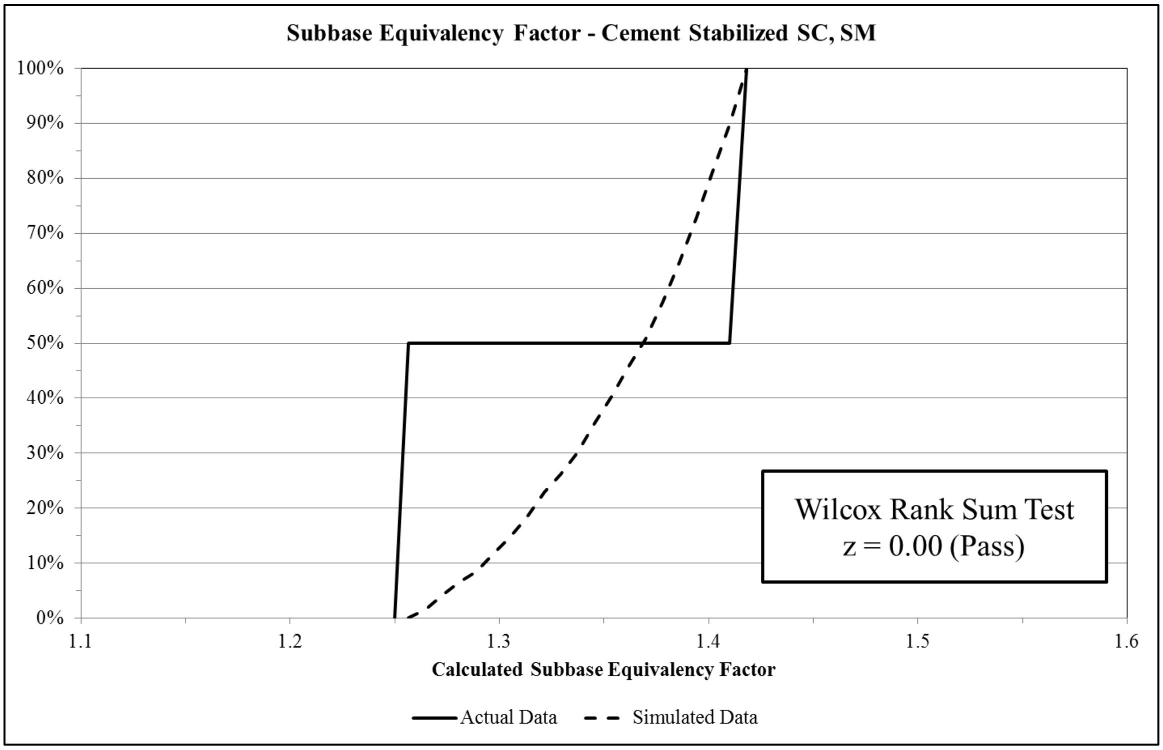


Figure 47. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Cement-Stabilized SC, SM (Subbase Course Equivalency Factor)

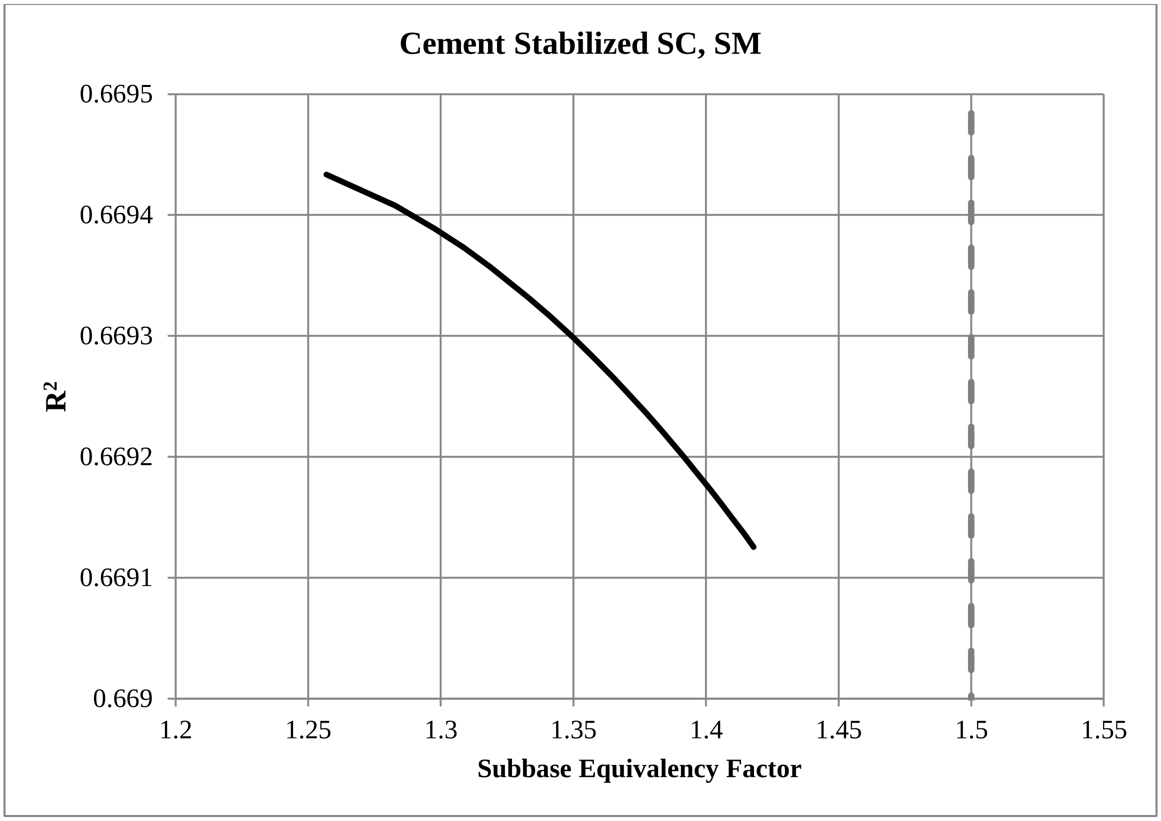


Figure 48. Optimization Curve for Cement-stabilized SC, SM Subbase Course Equivalency Factor

Lime-Stabilized ML, MH, CL, CH

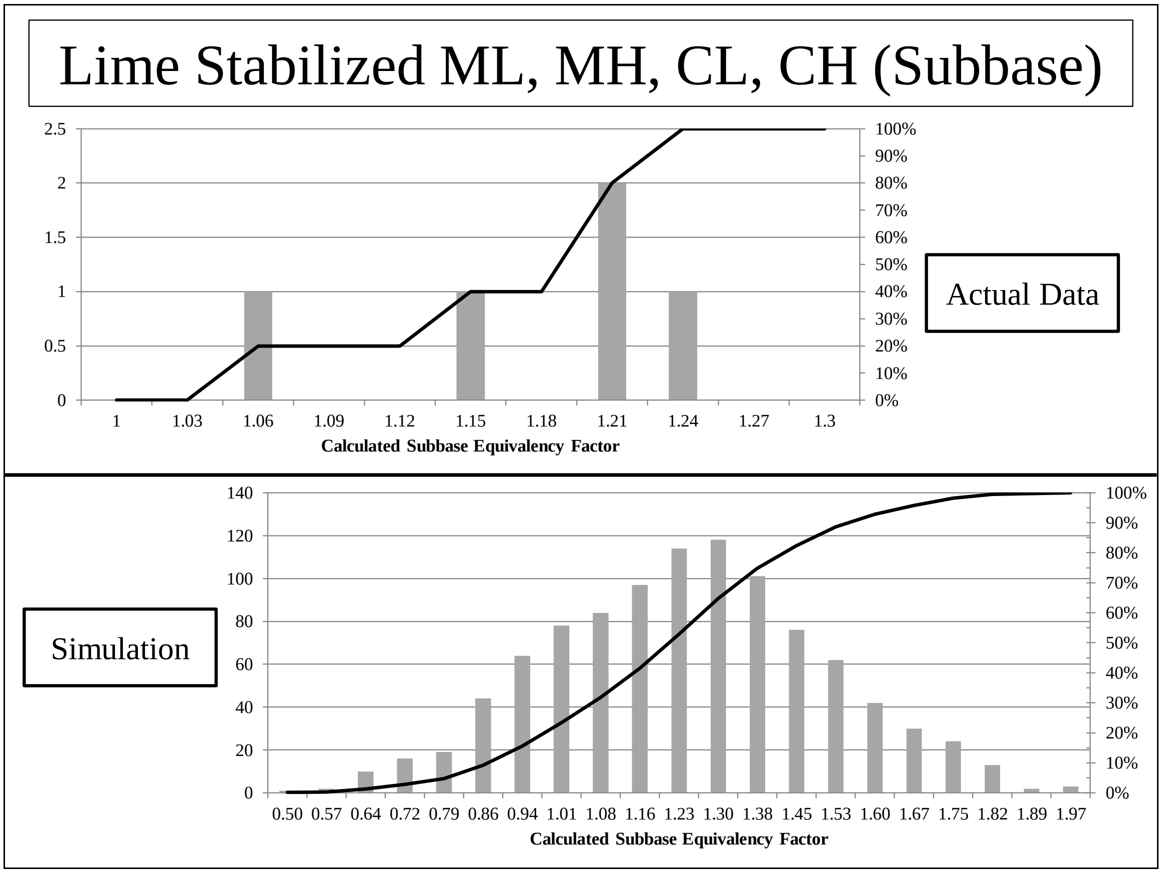


Figure 49. Comparison Between Actual and Simulated Data for Lime-Stabilized ML, MH, CL, CH (Subbase Course Equivalency Factor)

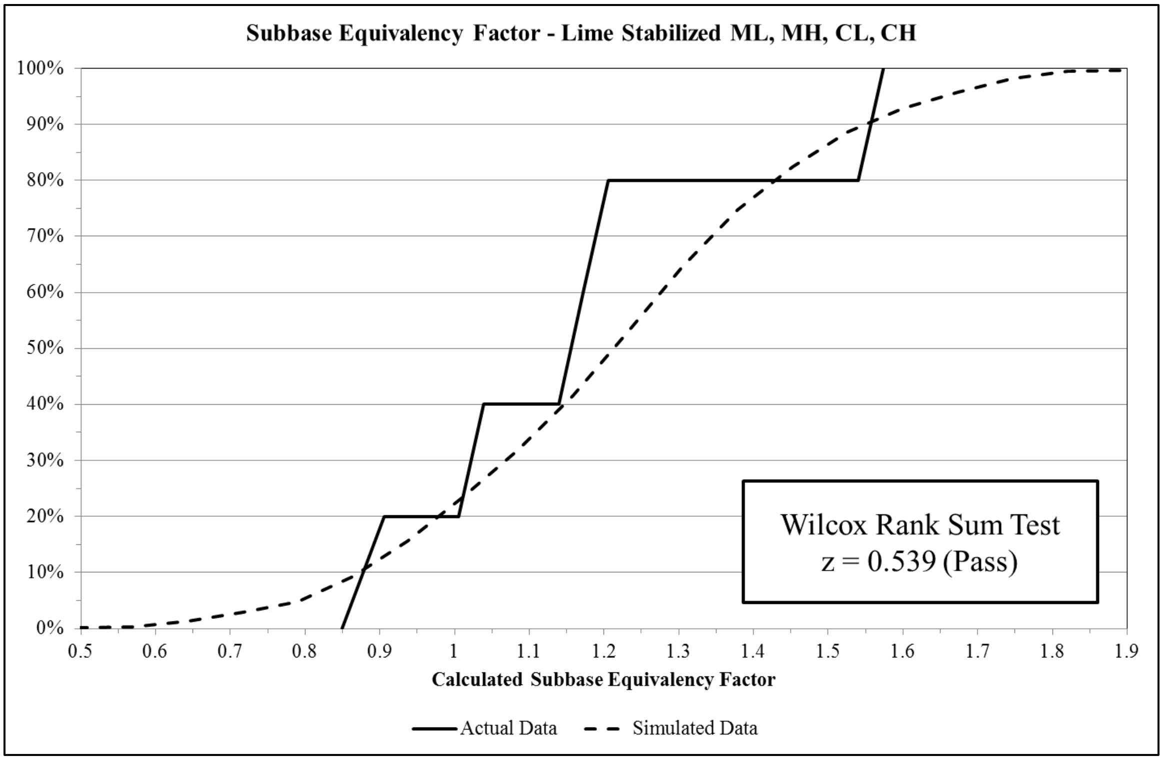


Figure 50. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Lime-Stabilized ML, MH, CL, CH (Subbase Course Equivalency Factor)

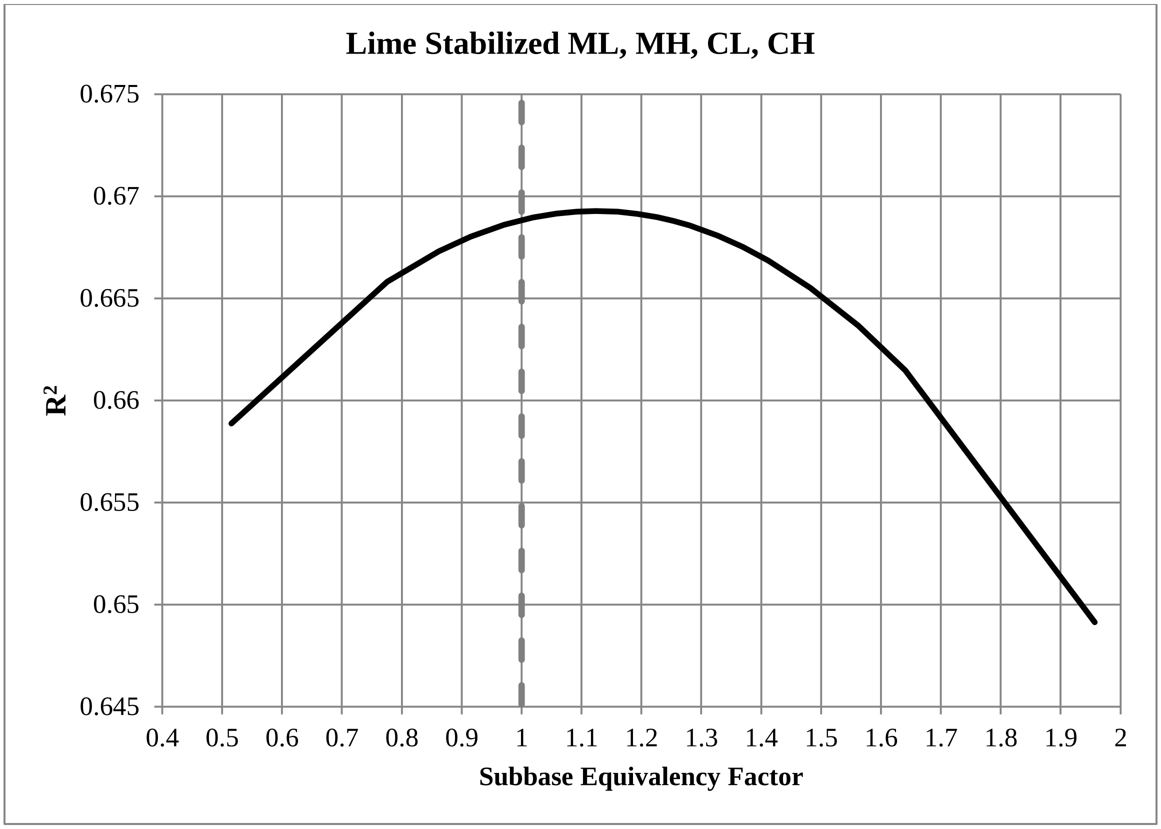


Figure 51. Optimization Curve for Lime Stabilized ML, MH, CL, CH Subbase Course Equivalency Factor

Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH

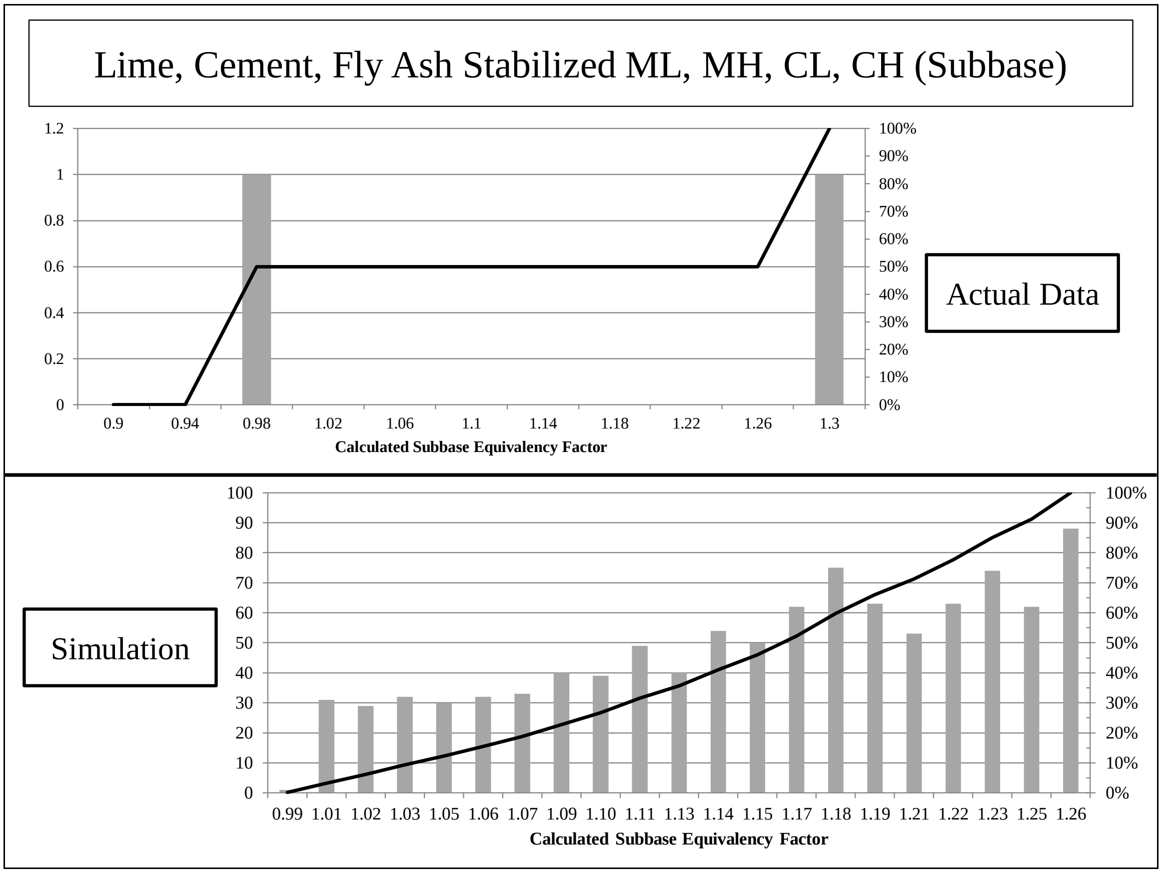


Figure 52. Comparison Between Actual and Simulated Data for Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH (Subbase Course Equivalency Factor)

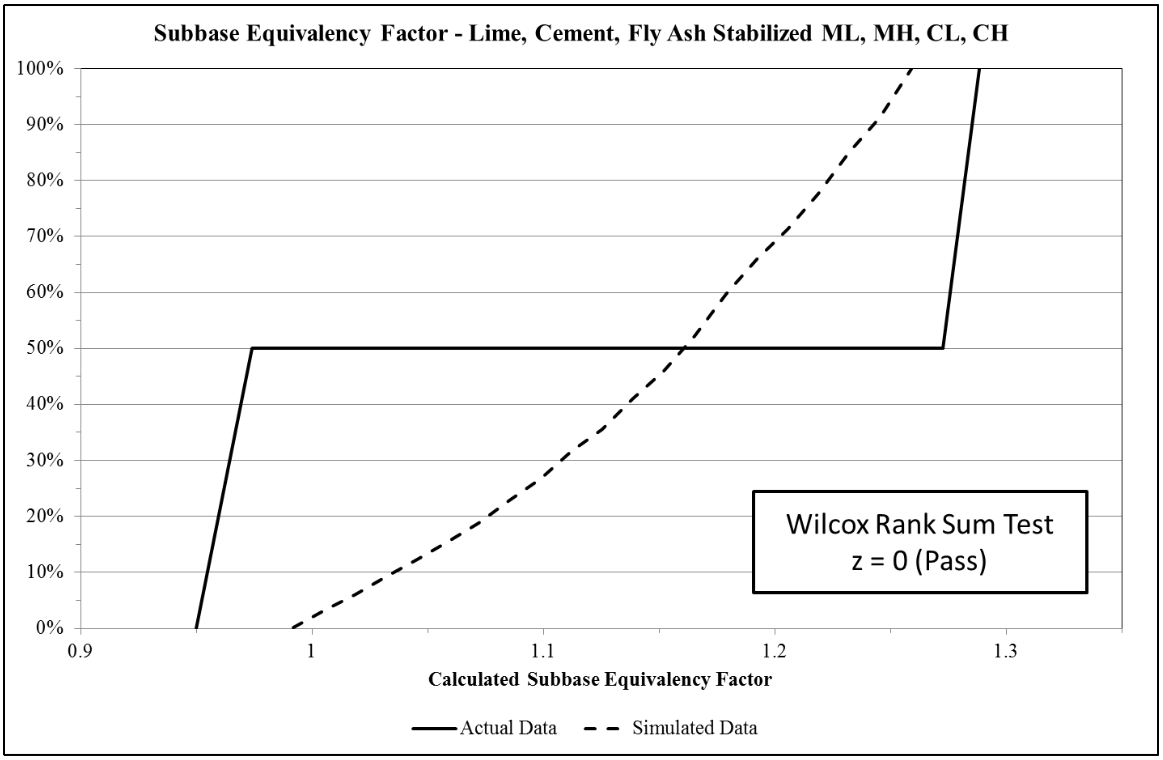


Figure 53. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH (Subbase Course Equivalency Factor)

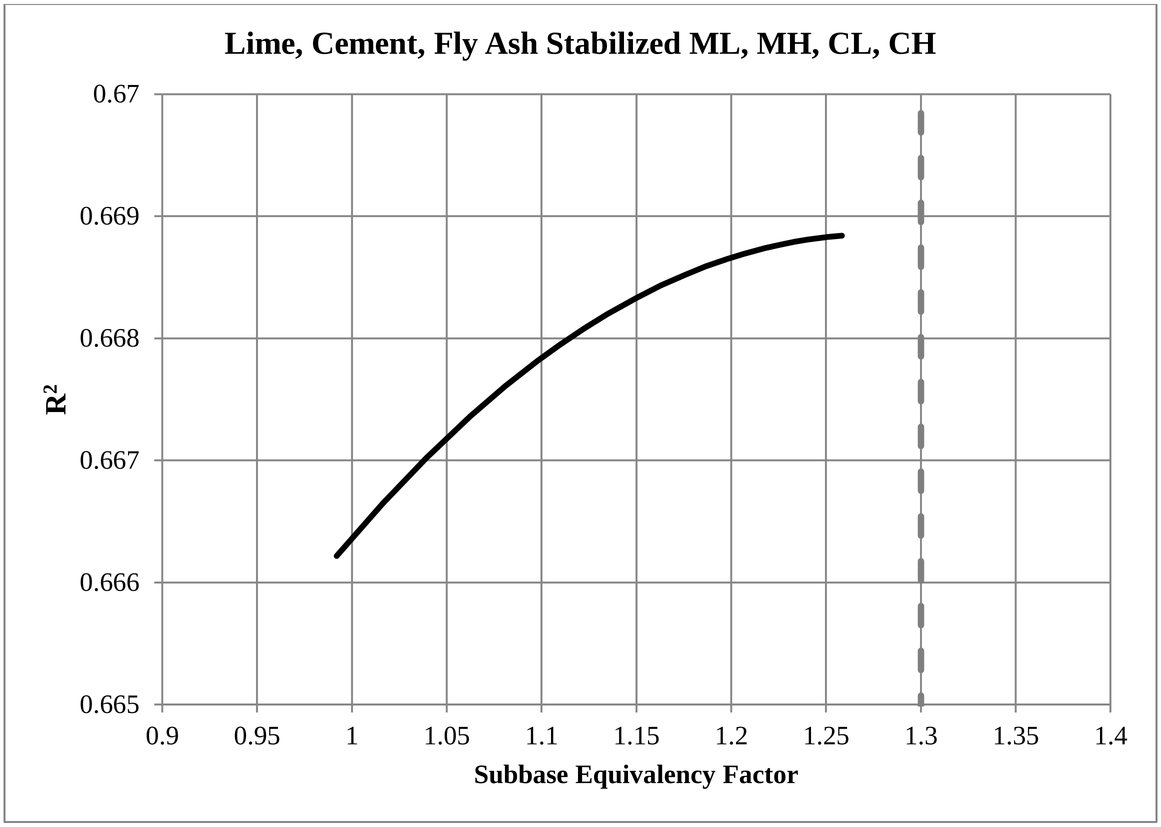


Figure 54. Optimization Curve for Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH Subbase Course Equivalency Factor

Crushed Aggregate (P-209)

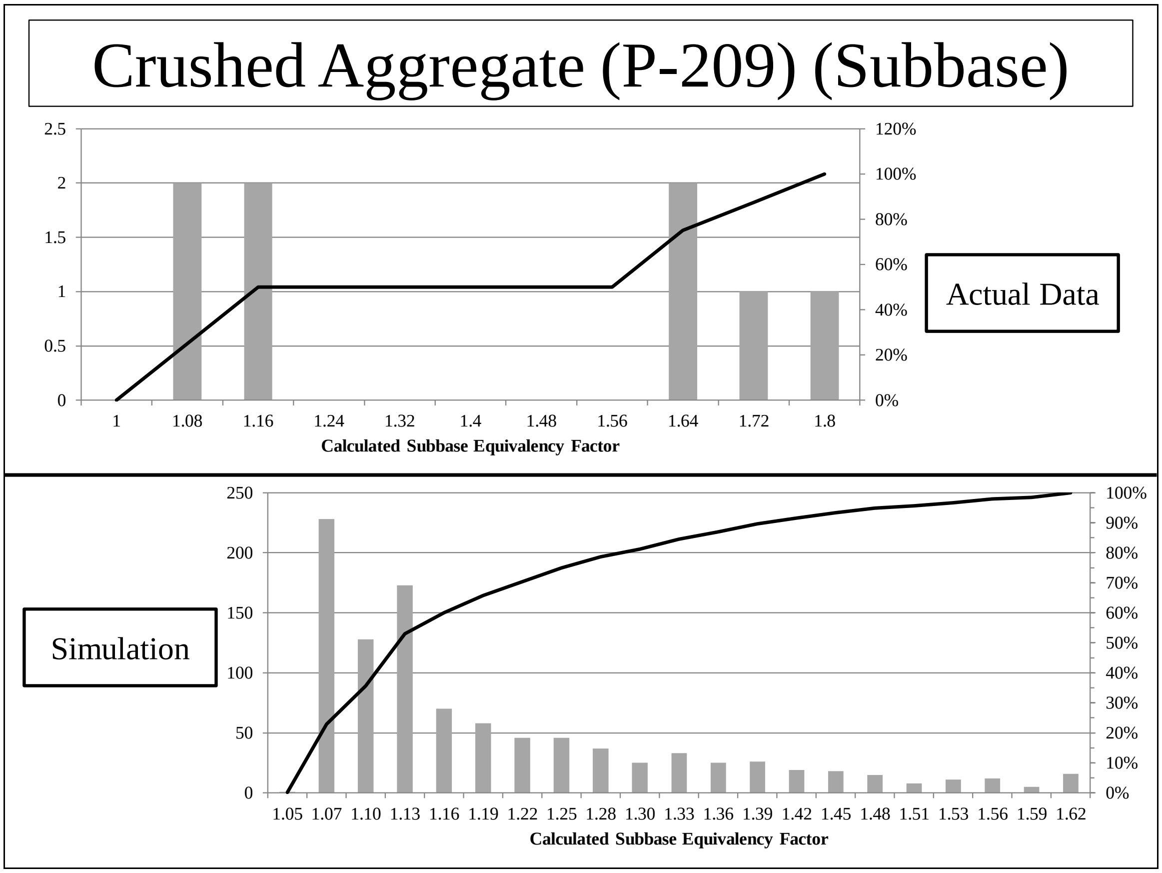


Figure 55. Comparison Between Actual and Simulated Data for Crushed Aggregate (Subbase Course Equivalency Factor)

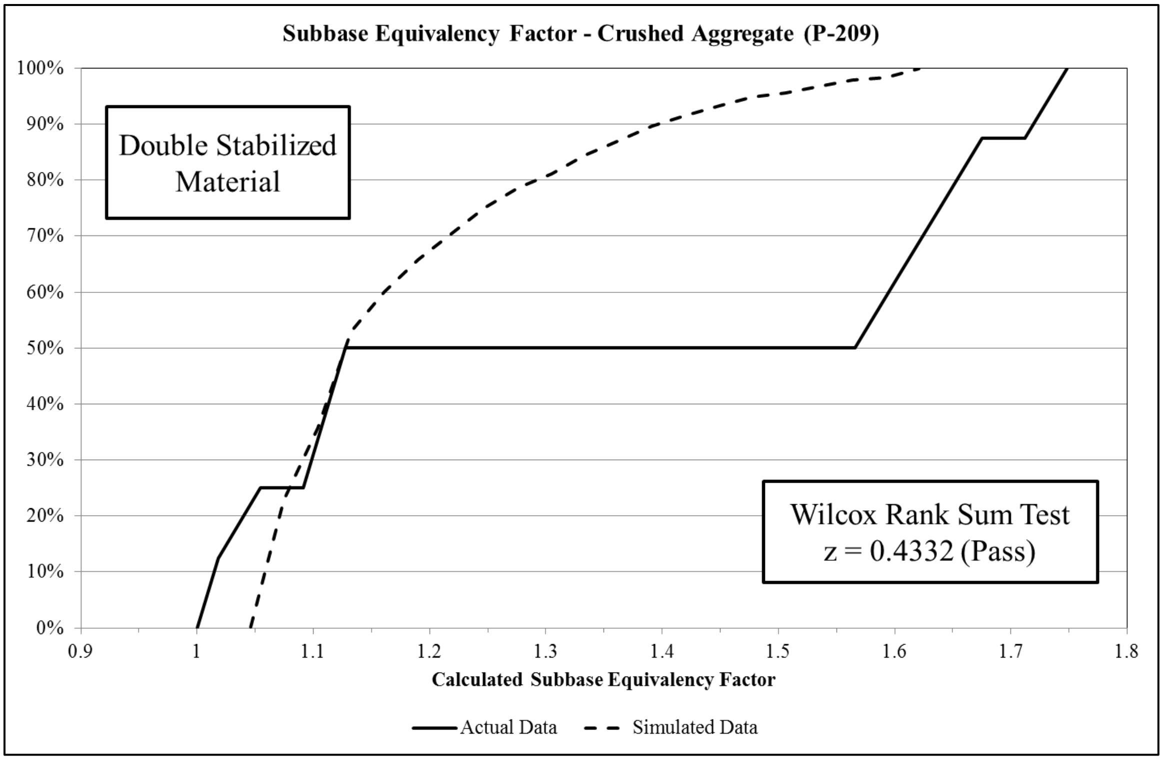


Figure 56. Comparison of the Cumulative Distributions for the Actual and Simulated Data for Crushed Aggregate (Subbase Course Equivalency Factor)

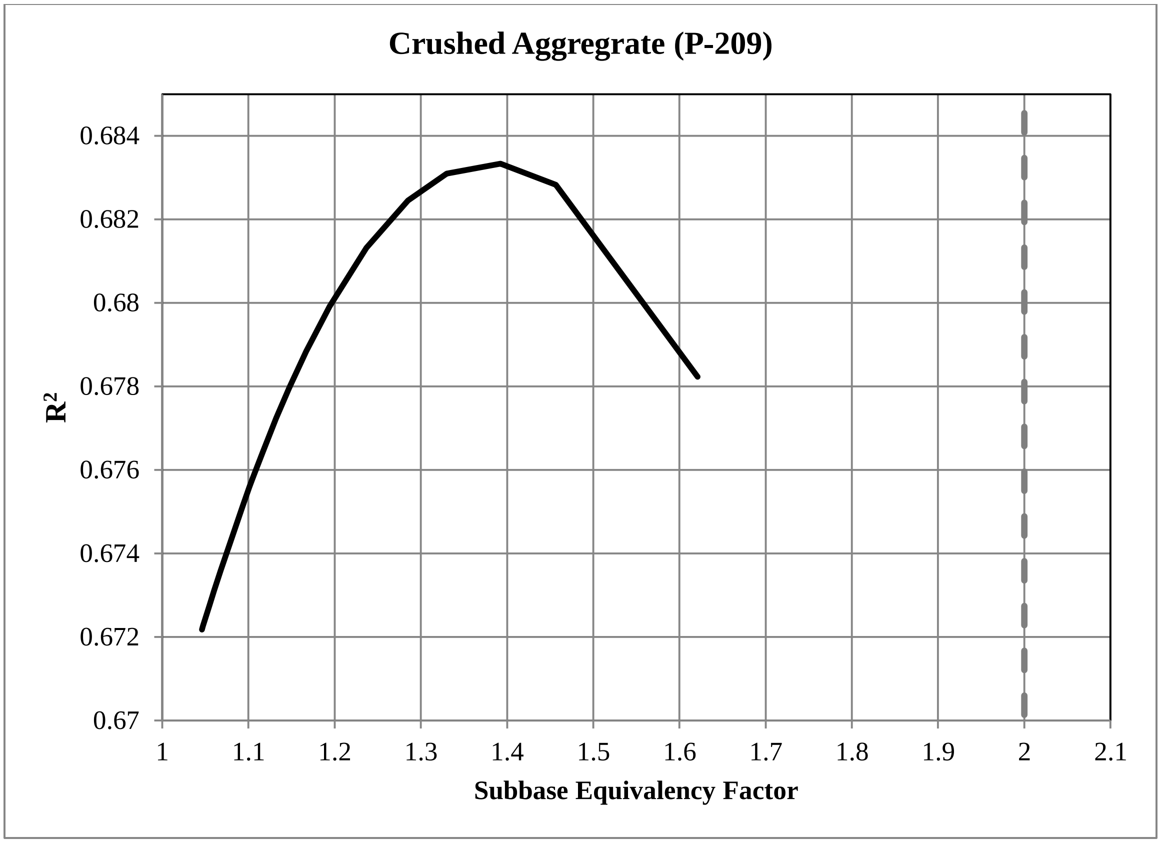


Figure 57. Optimization Curve for Crushed Aggregate (P-209) Subbase Course Equivalency Factor

Appendix B. Supporting Documentation for Cost Comparison between Various Flexible Airfield Pavement Design Methodologies

The cost comparison of the various flexible airfield pavement design methods utilized 81 standard design scenarios using the variables shown in Table 20. To develop a scenario, the research team chose one variable from each category in Table 20 and then designed the pavement using these inputs. This process was repeated 81 times to run the scenario using every possible combination of the variables. The three aircraft were chosen because the F-15E, C-17, and the 777-300 were common amongst the design methods and represented potential aircraft that would utilize military airfields. Additionally, these aircraft represented the few aircraft with historical test section data.

Table 20. Variables used in Cost Comparison to Develop 81 Scenarios

Aircraft	Base Construction	Passes	Subgrade CBR
F-15E	Conventional (80 CBR)	1,000	3
C-17	Cement-Stabilized	10,000	6
777-300	Asphalt-Stabilized	100,000	10

Unit Cost Data

The unit cost data for this analysis came from RS Means's *Site Work and Landscape Work*. Since the cost data were given in 2010 dollars, the research used the Consumer Price Index (CPI) to convert the unit costs to current dollar values. At the time of this study, the annual average CPI was not available for 2013; therefore, the CPI value of 234.149 for September was used for this study. The annual average CPI value for 2010 was 218.056 (U.S. Department of Labor, 2014).

As shown in Table 21, the research team considered only the costs associated with the five items listed in the table. Each of these line items included the costs of the material and necessary installation processes, such as compacting. Any additional line items that may be necessary for construction of the pavement section were not included in the study as it was considered common among all the methods for a given design scenario. Additionally, the costs were evaluated for a 200 feet by 2,000 feet pavement section.

Table 21. Unit Cost Data for Life-Cycle Cost Analysis of Various Flexible Design Methods (RS Means, 2010)

	Years		
	2010	**2013**	**Units**
Asphalt	$4.80	$5.15	Per Square Yard Per Inch
Subbase Course	$0.90	$0.97	Per Square Yard Per Inch
Base Course	$1.54	$1.66	Per Square Yard Per Inch
Cement-Stabilized Base Course	$2.64	$2.84	Per Square Yard Per Inch
Asphalt-stabilized Base Course	$2.58	$2.77	Per Square Yard Per Inch

Design Aircraft Results

Boeing F-15E Strike Eagle

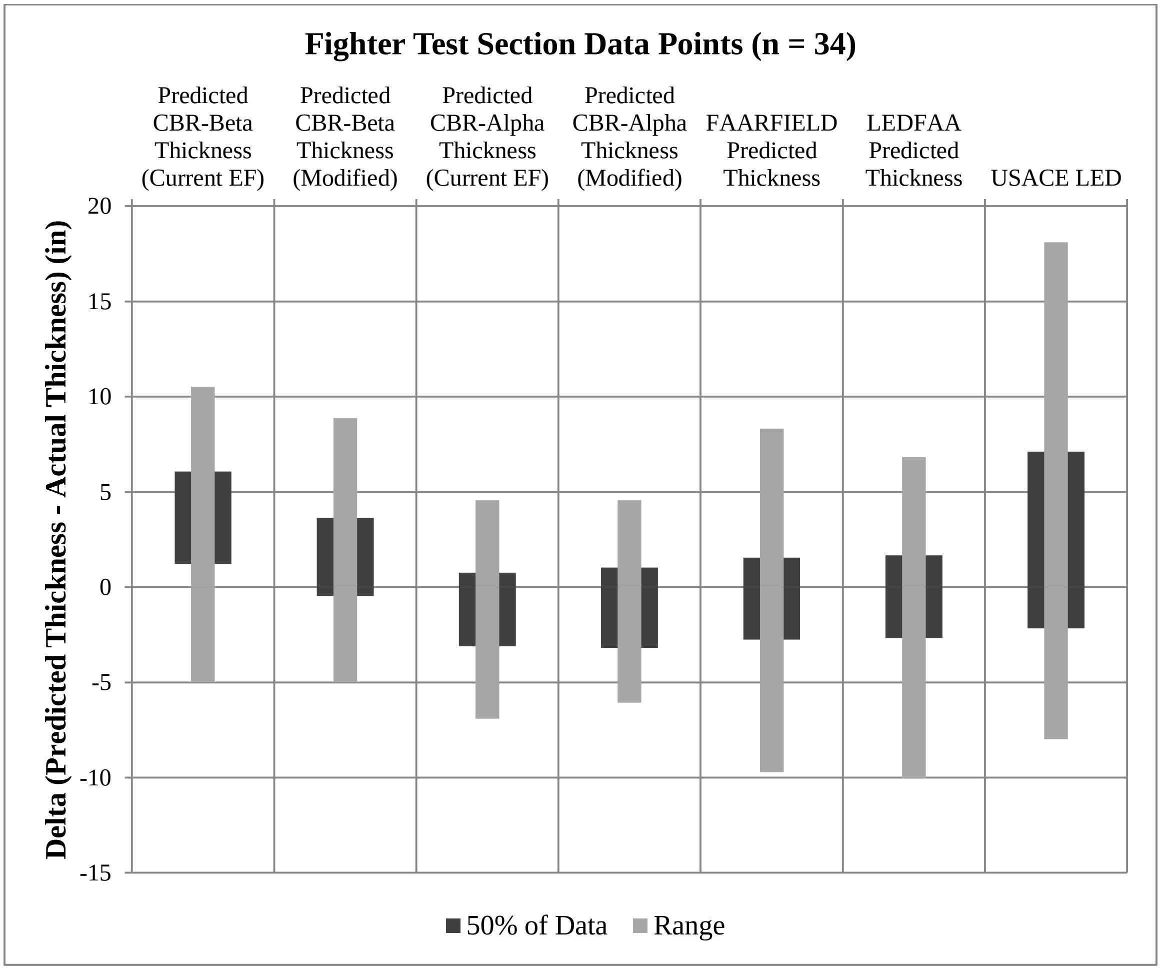


Figure 58. Comparison of the Relative Error for the Various Flexible Design Methods at Predicting the Performance of Historical Test Sections – F-15E

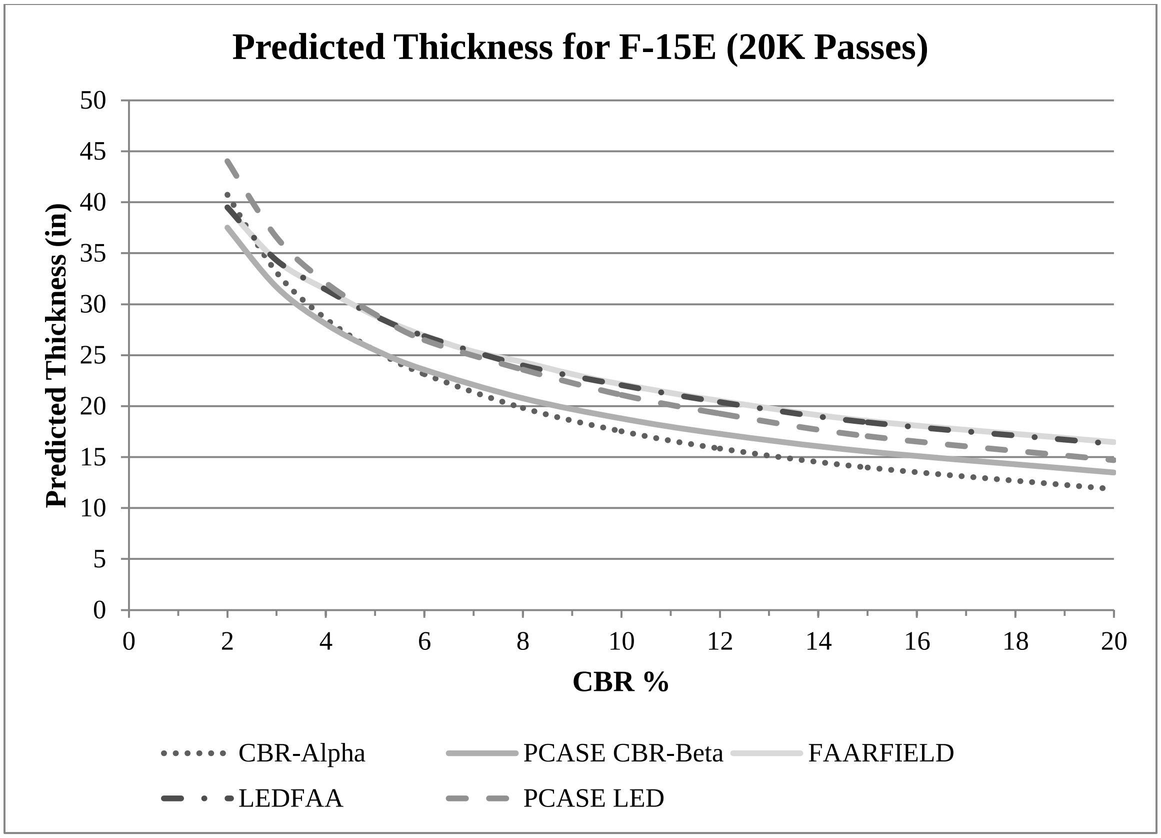


Figure 59. Comparison of the Predicted Thicknesses for the Various Flexible Pavement Design Methods for the F-15E at 20,000 Passes with Varying Subgrade CBR

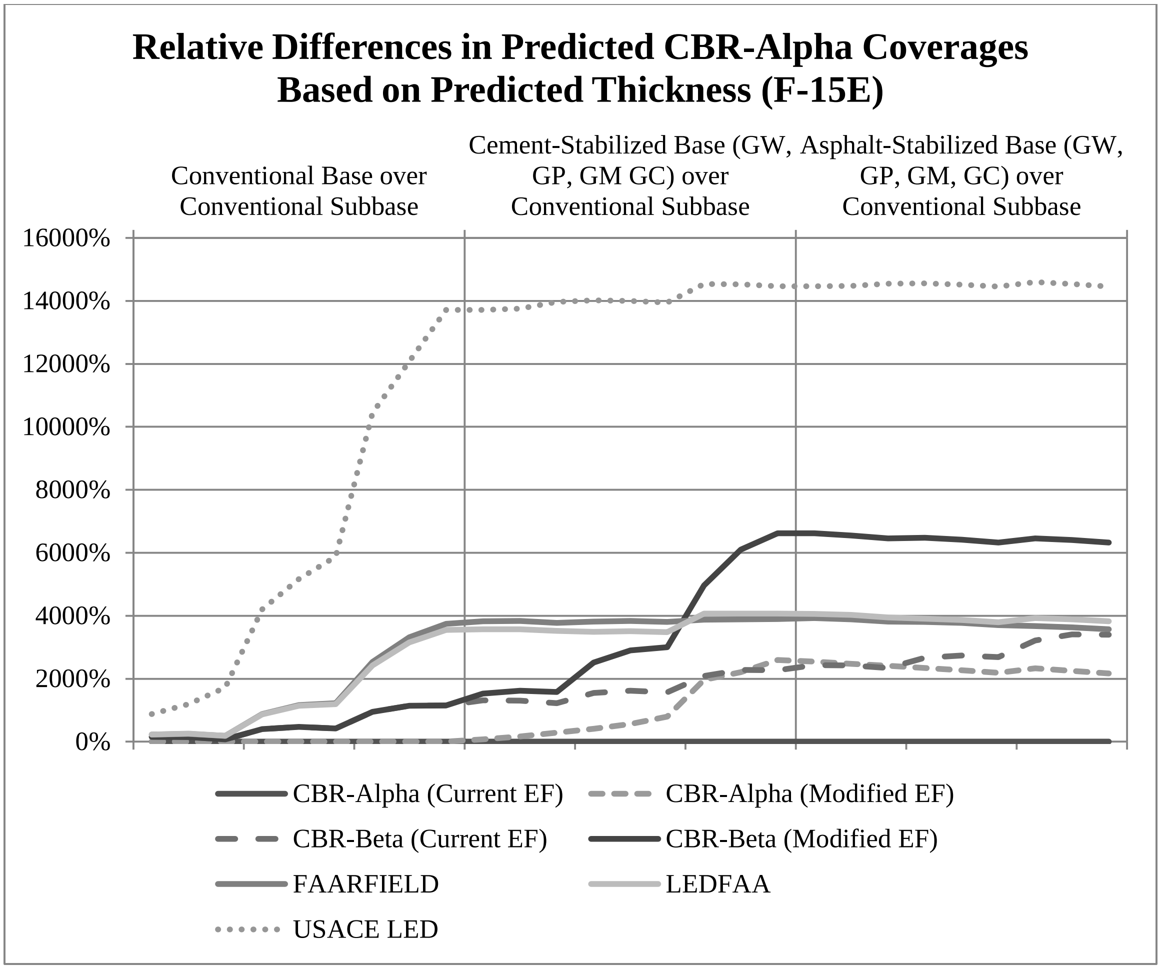


Figure 60. Comparison of the Cumulative Differences in Predicted CBR-Alpha Coverages Based on the Predicted Thickness Calculated by the Various Flexible Pavement Design Methods – F-15E (n = 27)

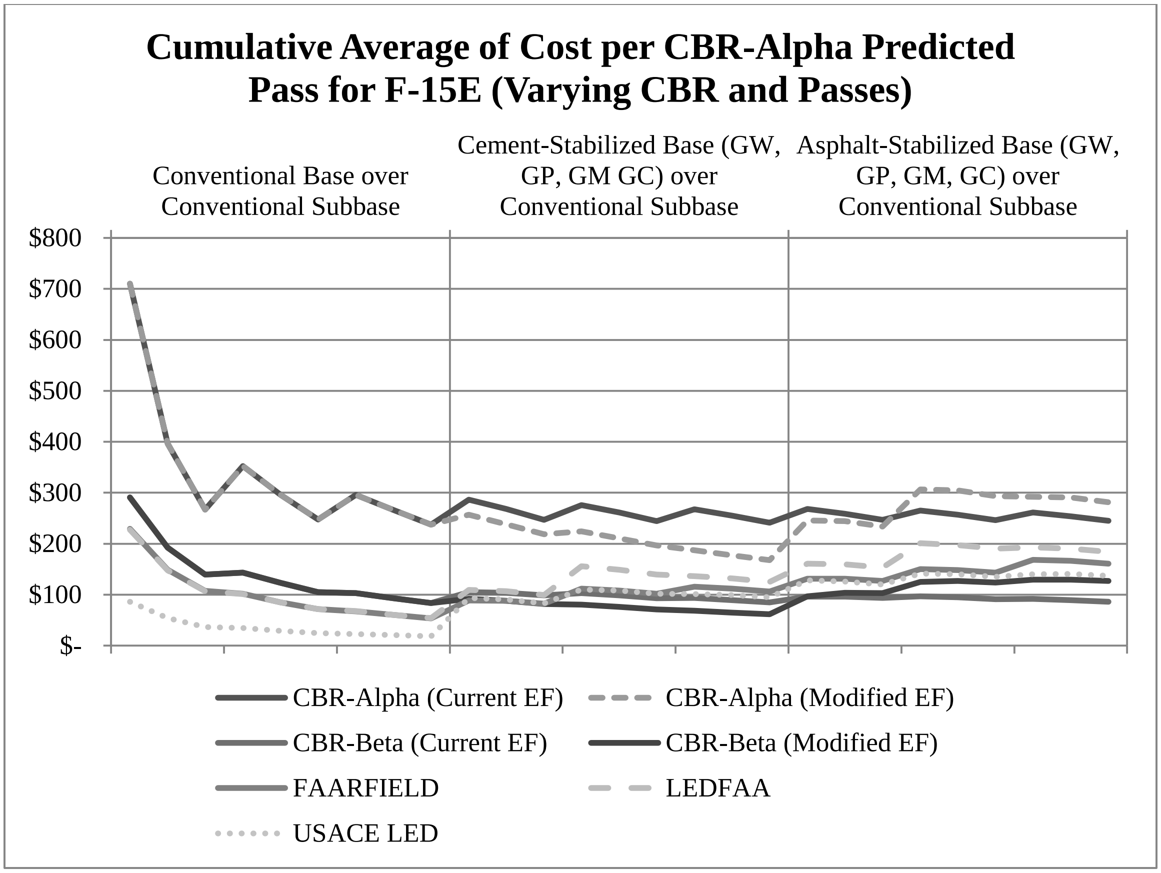


Figure 61. Comparison of the Cumulative Averages of the Cost per CBR-Alpha Predicted Pass Based on the Predicted Thickness Calculated by the Various Flexible Pavement Design Methods – F-15E (n = 27)

Boeing C-17 Globemaster III

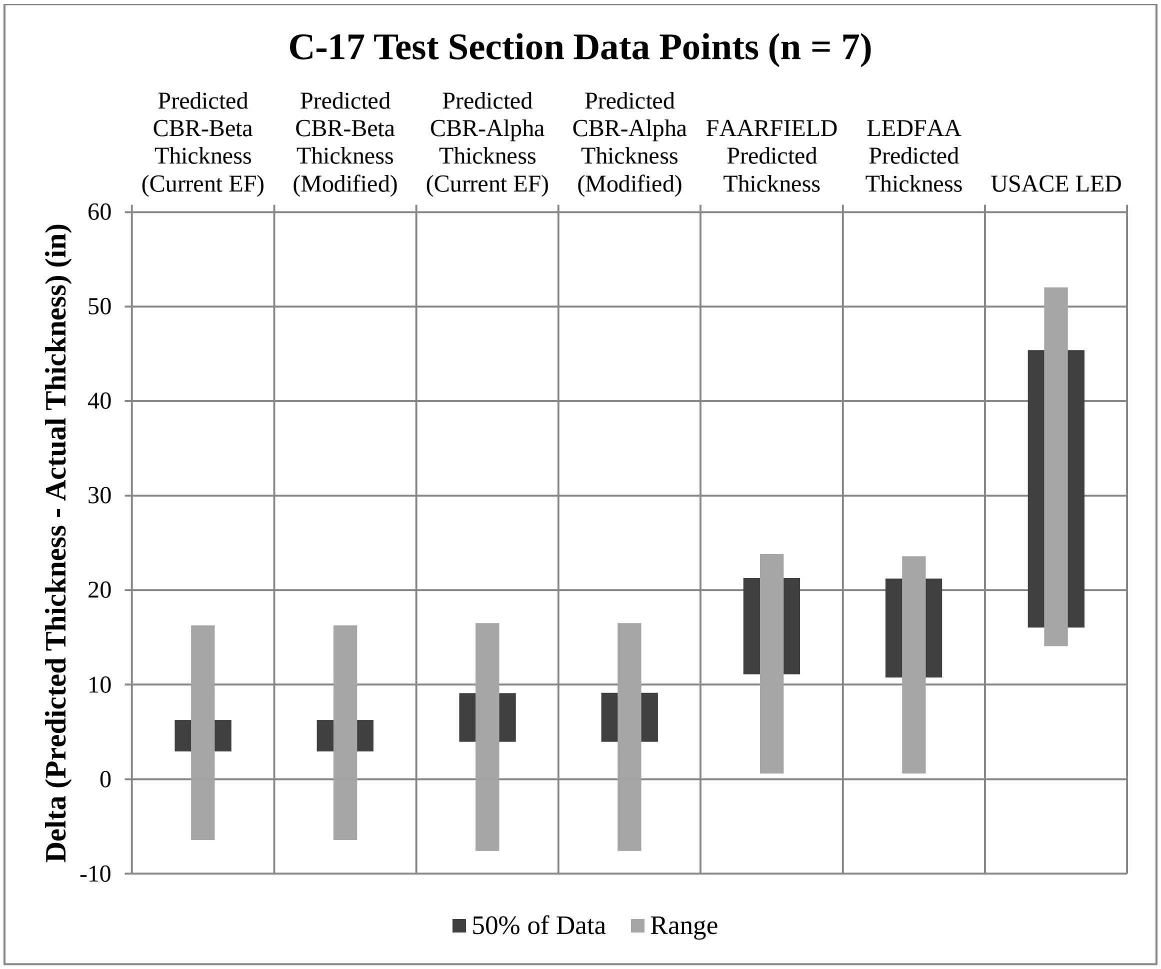


Figure 62. Comparison of the Relative Error for the Various Flexible Design Methods at Predicting the Performance of Historical Test Sections – C-17

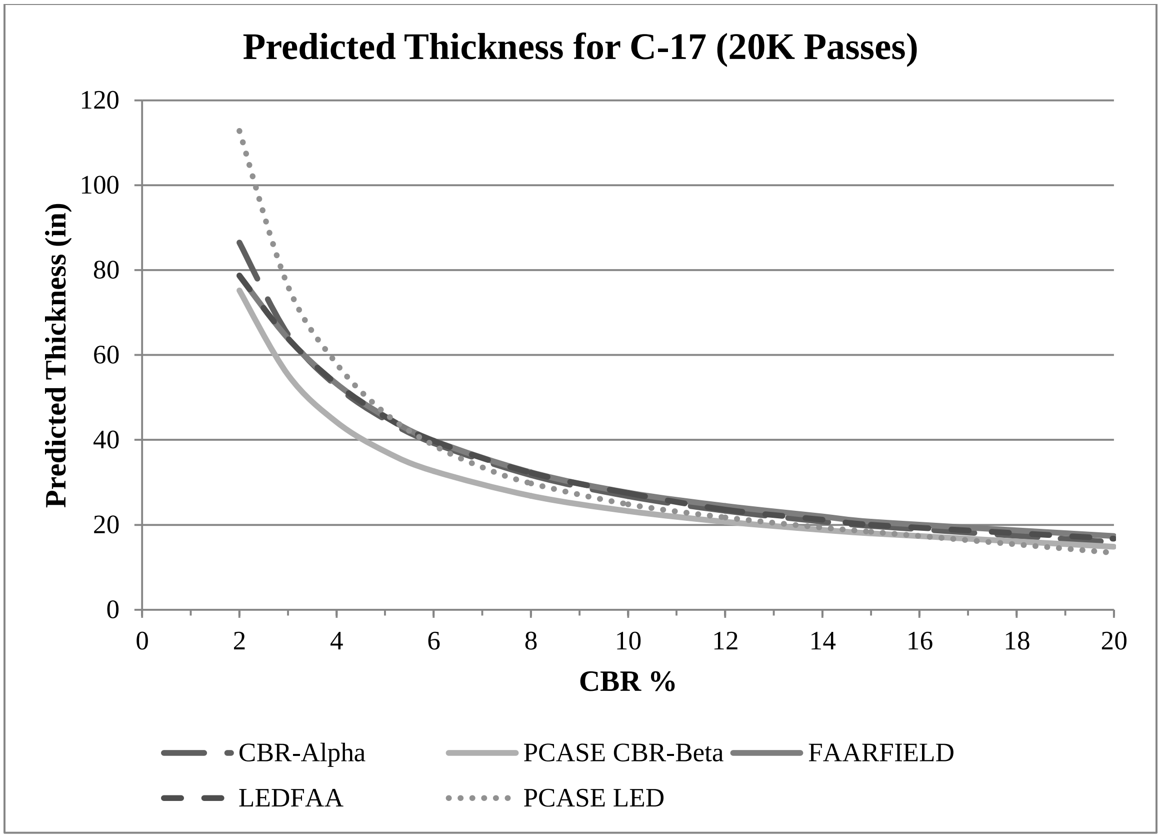


Figure 63. Comparison of the Predicted Thicknesses for the Various Flexible Pavement Design Methods for the C-17 at 20,000 Passes with Varying Subgrade CBR

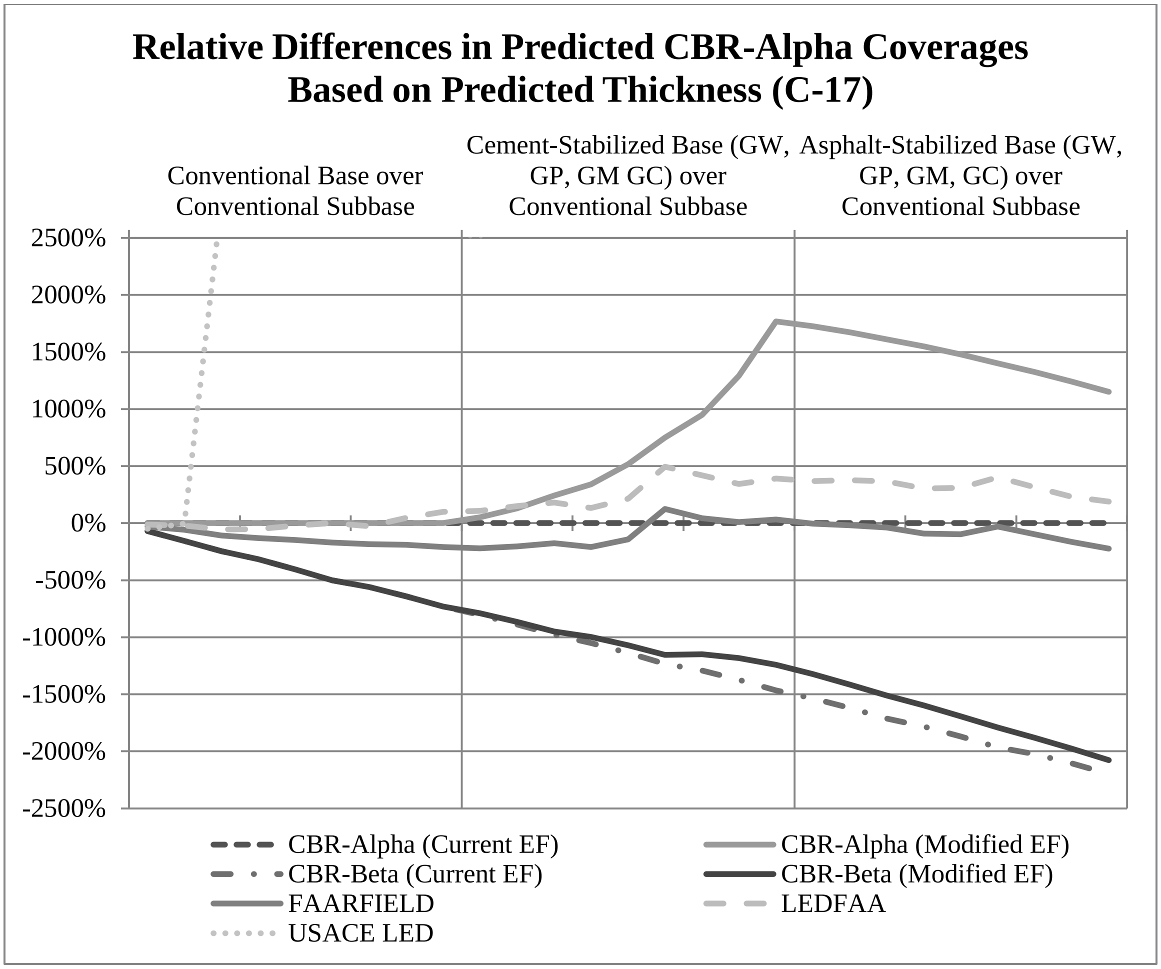


Figure 64. Comparison of the Cumulative Differences in Predicted CBR-Alpha Coverages Based on the Predicted Thickness Calculated by the Various Flexible Pavement Design Methods – C-17 (n = 27)

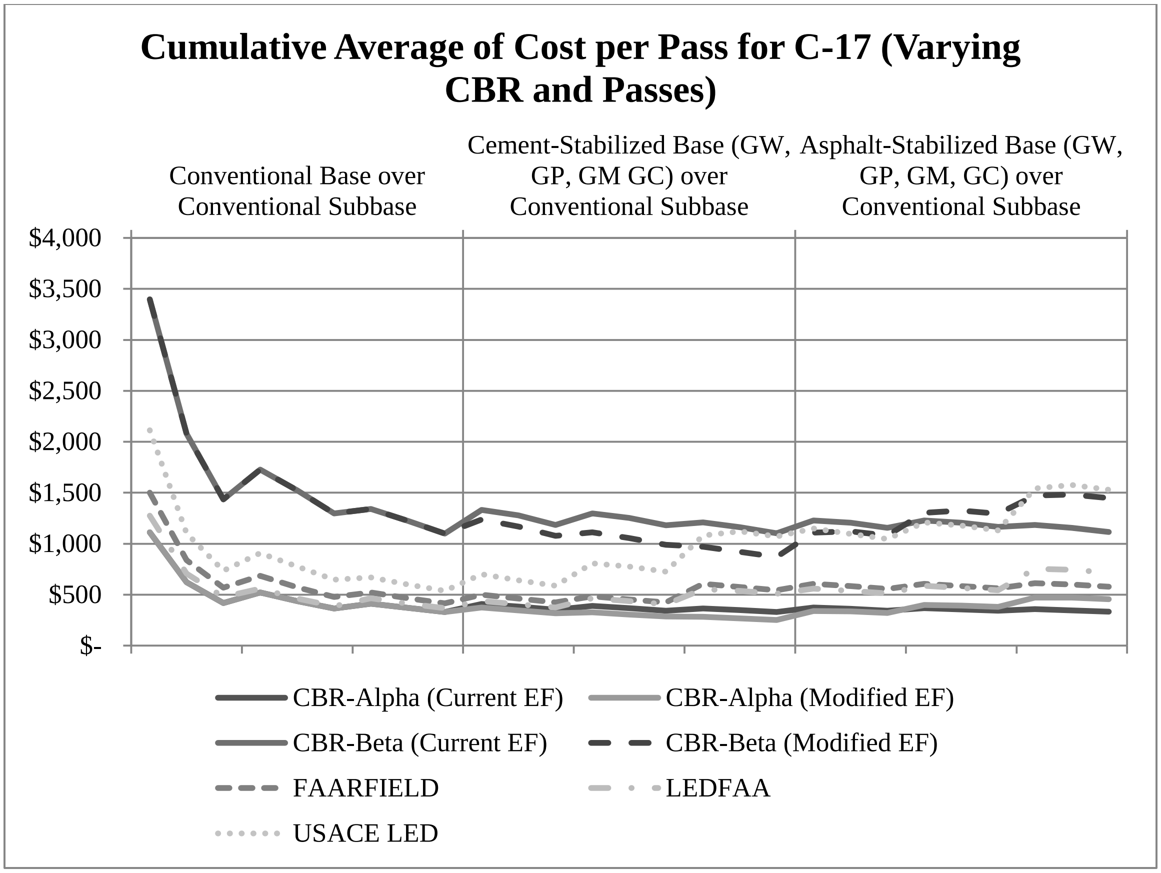


Figure 65. Comparison of the Cumulative Averages of the Cost per CBR-Alpha Predicted Pass Based on the Predicted Thickness Calculated by the Various Flexible Pavement Design Methods – C-17 (n = 27)

Boeing 777-300

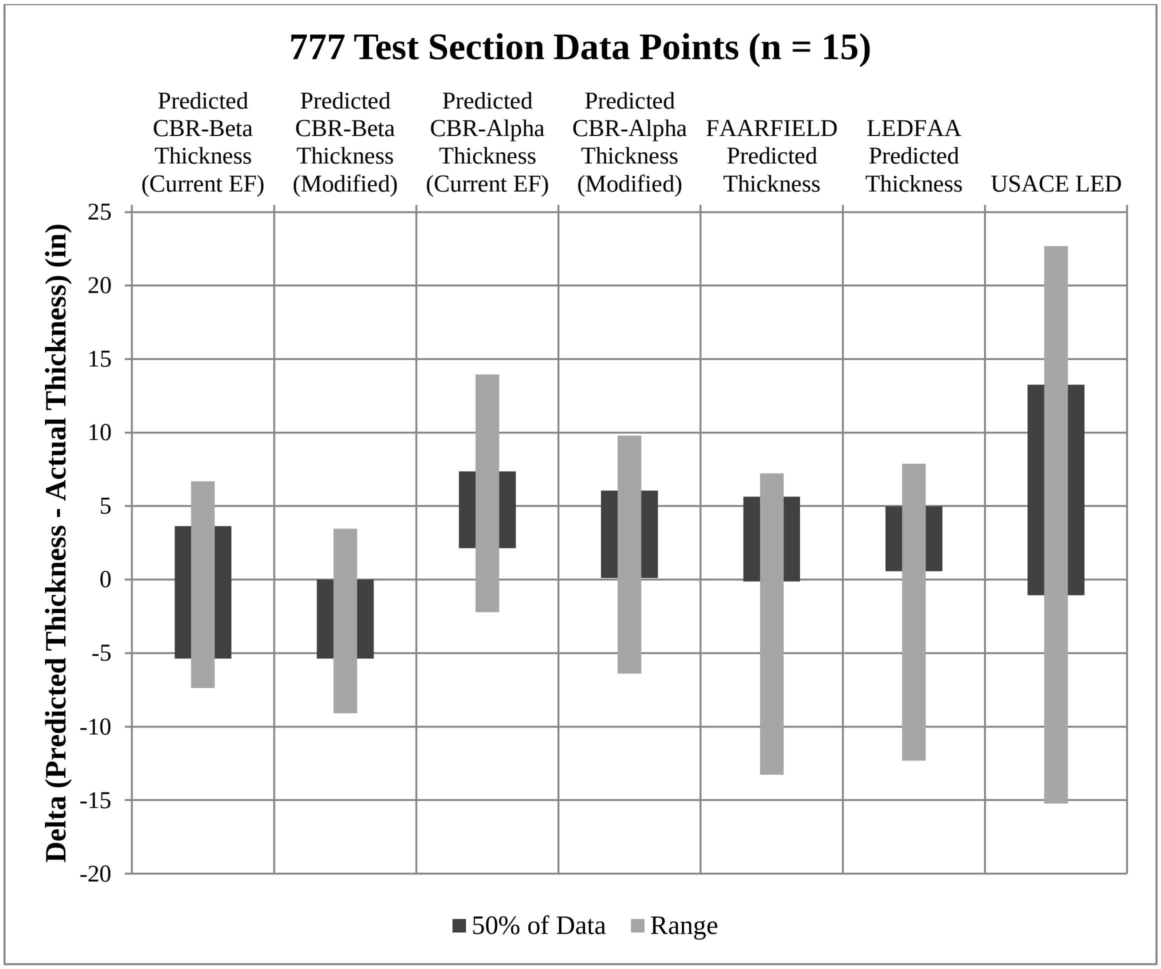


Figure 66. Comparison of the Relative Error for the Various Flexible Design Methods at Predicting the Performance of Historical Test Sections – Boeing 777

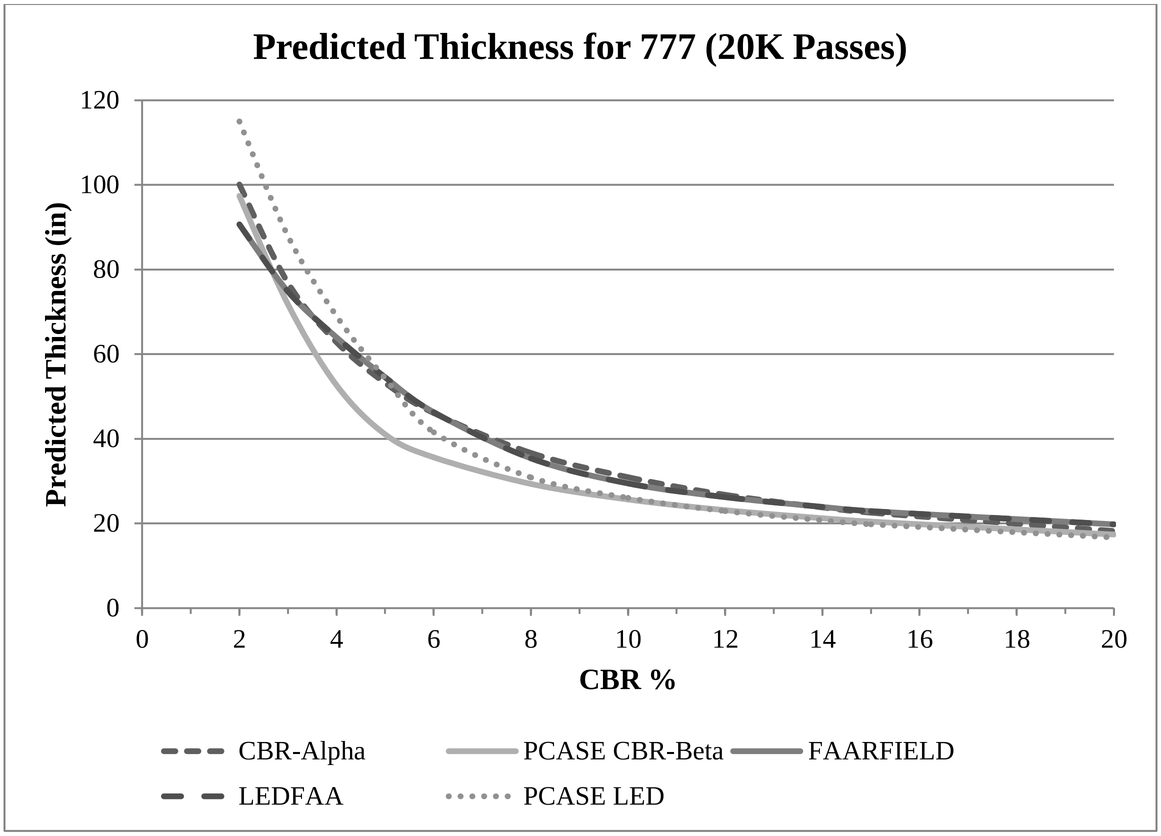


Figure 67. Comparison of the Predicted Thicknesses for the Various Flexible Pavement Design Methods for the Boeing 777 at 20,000 Passes with Varying Subgrade CBR

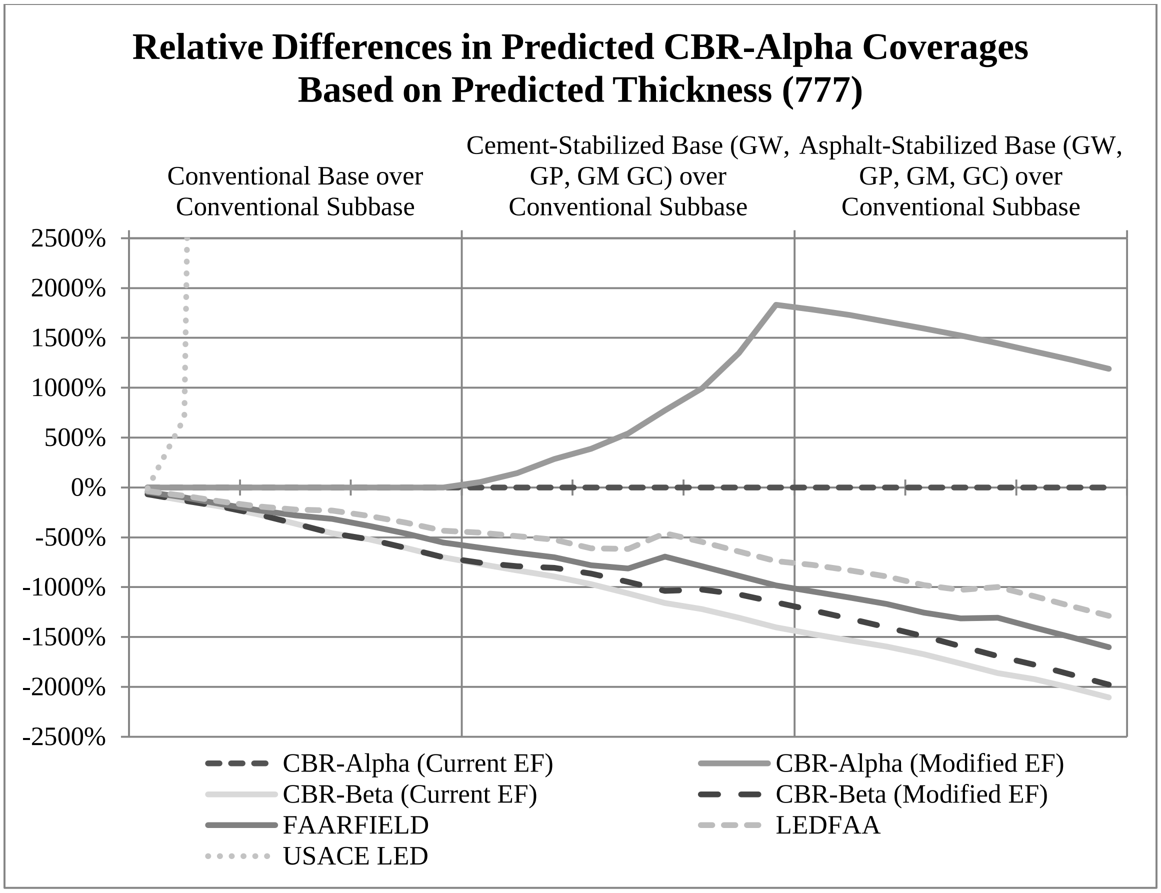


Figure 68. Comparison of the Cumulative Differences in Predicted CBR-Alpha Coverages Based on the Predicted Thickness Calculated by the Various Flexible Pavement Design Methods – Boeing 777 (n = 27)

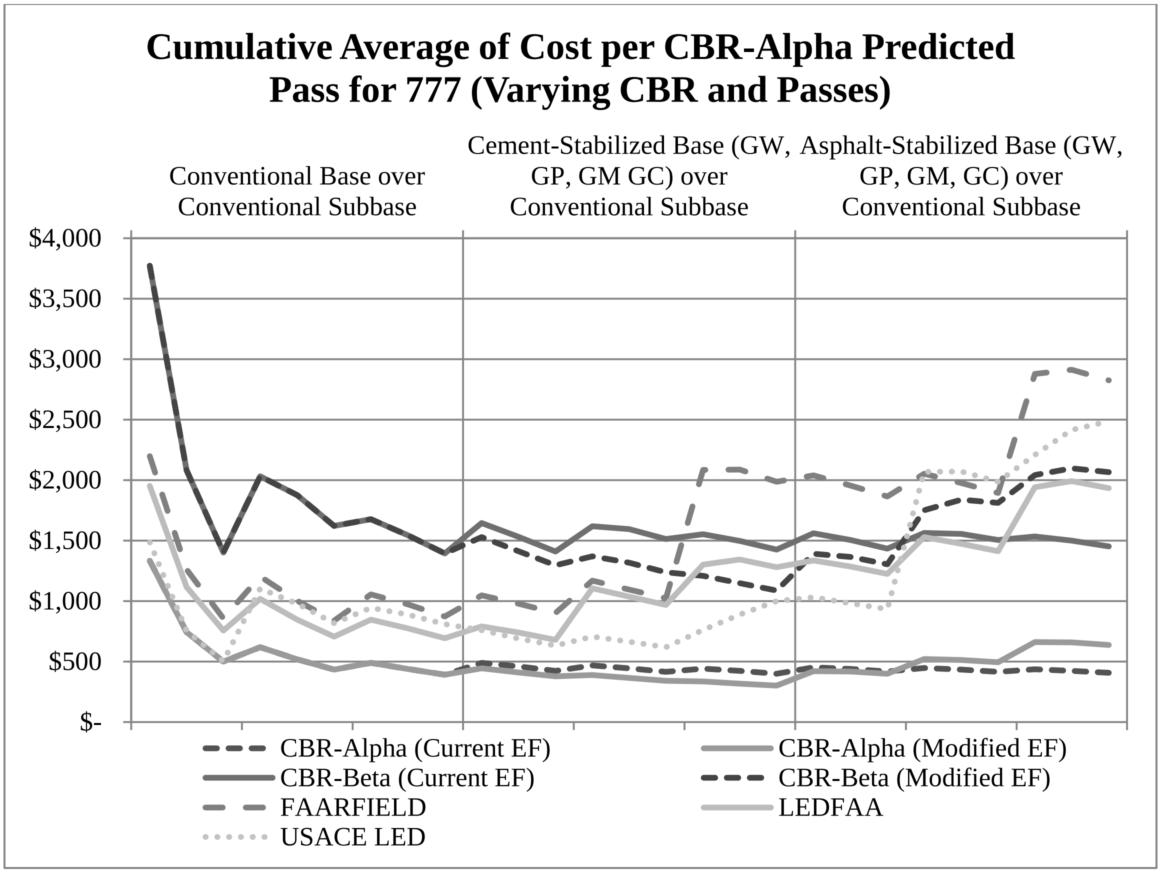


Figure 69. Comparison of the Cumulative Averages of the Cost per CBR-Alpha Predicted Pass Based on the Predicted Thickness Calculated by the Various Flexible Pavement Design Methods – Boeing 777 (n = 27)

Aggregated Results

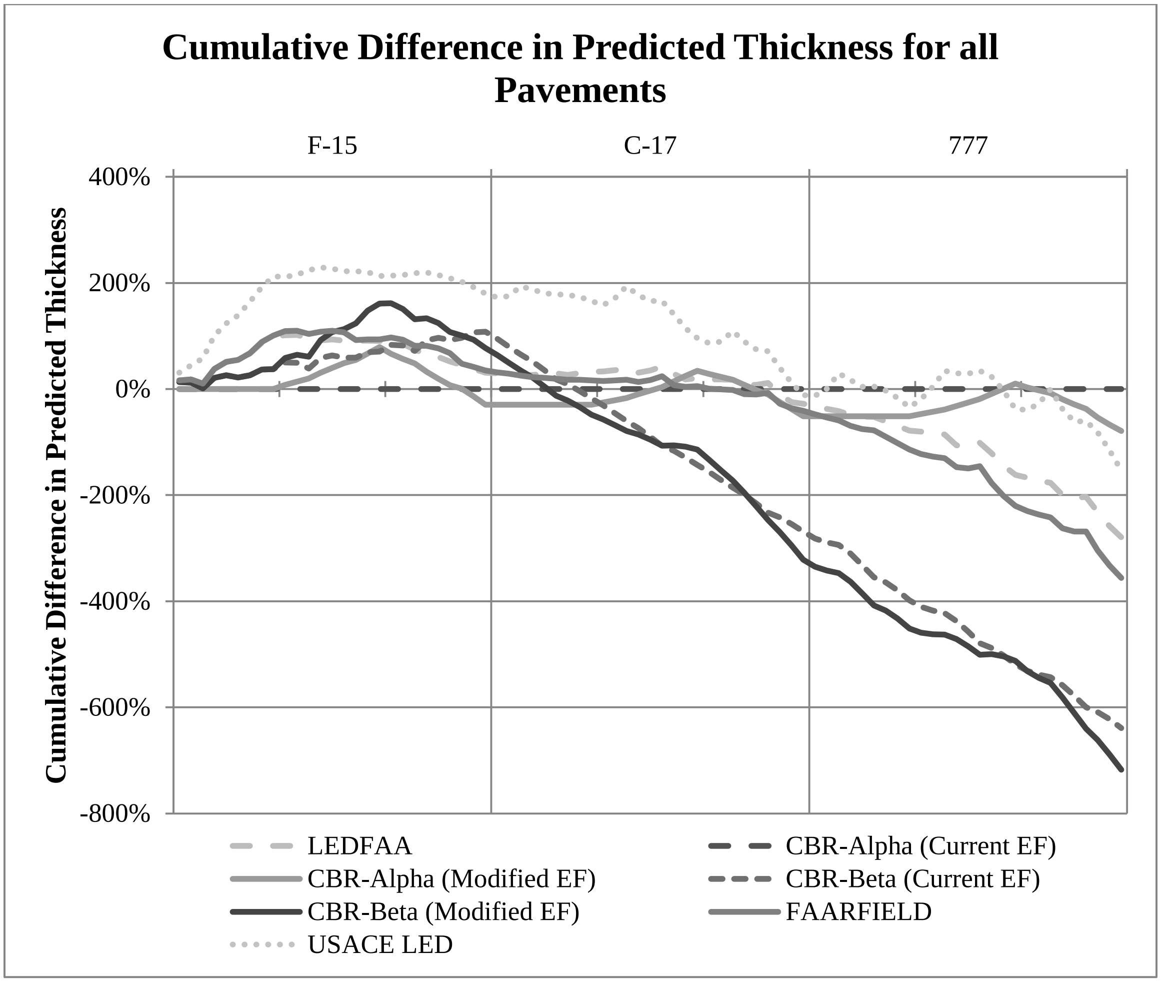


Figure 70. Comparison of the Cumulative Differences in Predicted Thickness for the Various Flexible Pavement Design Methods – Aggregated Results (n = 81)

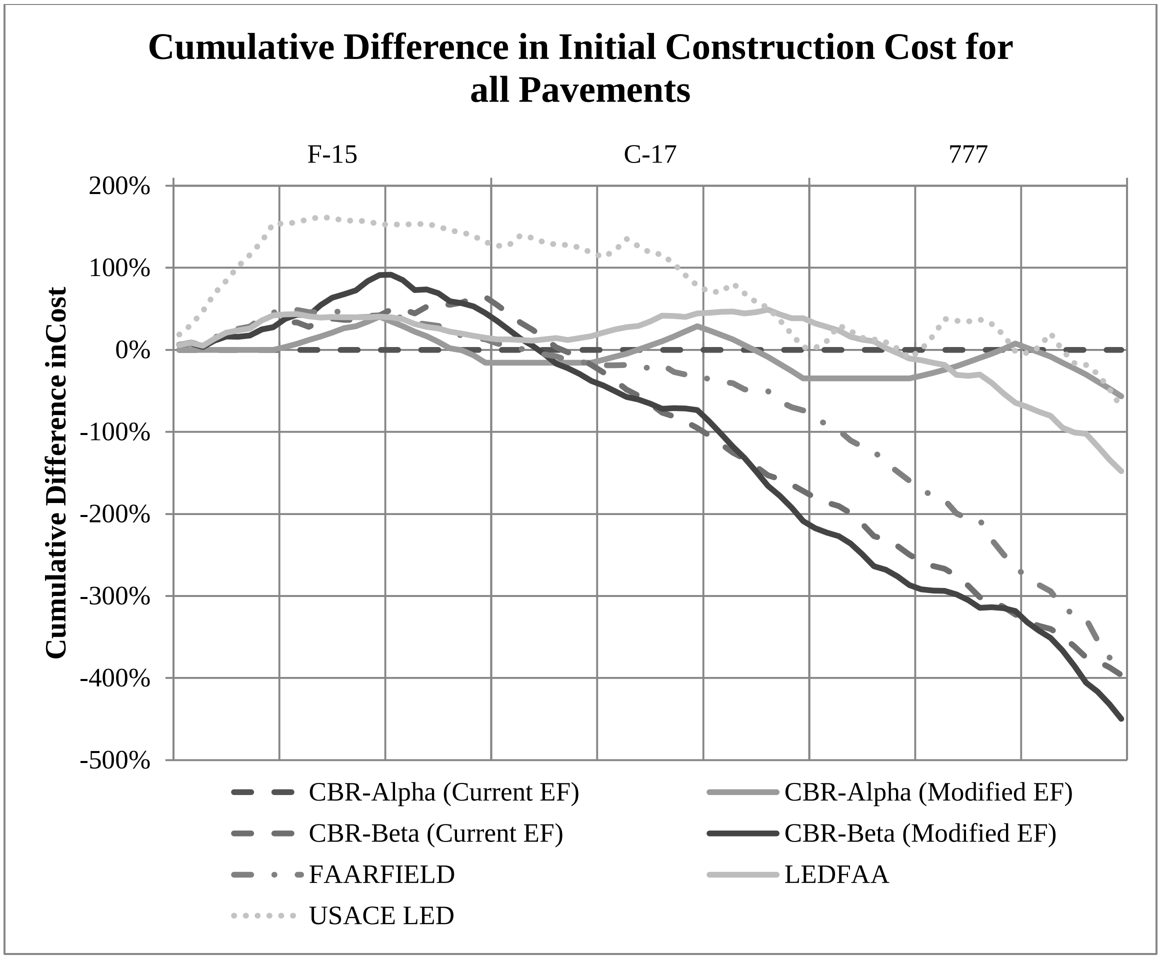


Figure 71. Comparison of the Cumulative Differences in Initial Construction Cost for the Various Flexible Pavement Design Methods – Aggregated Results (n = 81)

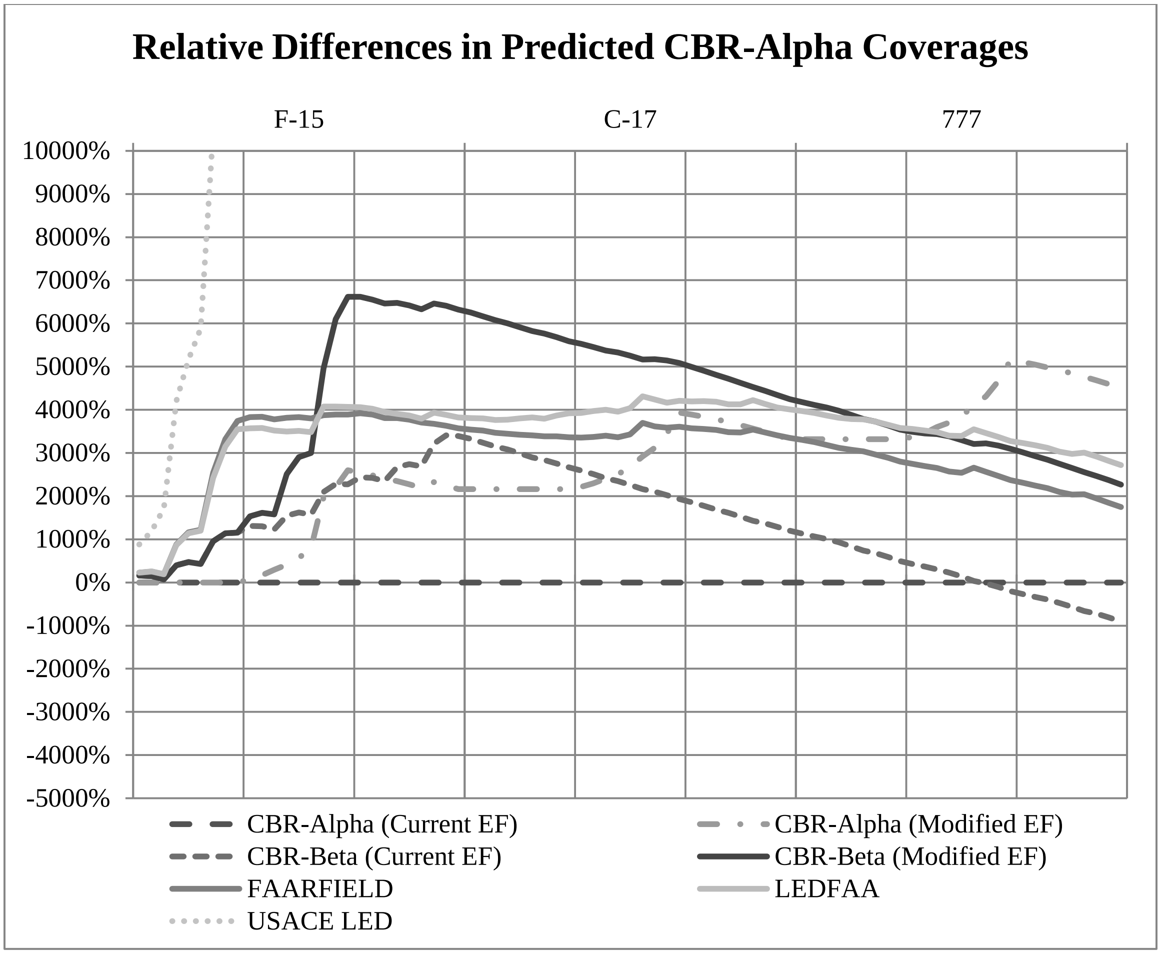


Figure 72. Comparison of the Cumulative Differences in Predicted CBR-Alpha Coverages Based on the Predicted Thickness Calculated by the Various Flexible Pavement Design Methods – Aggregated Results (n = 81)

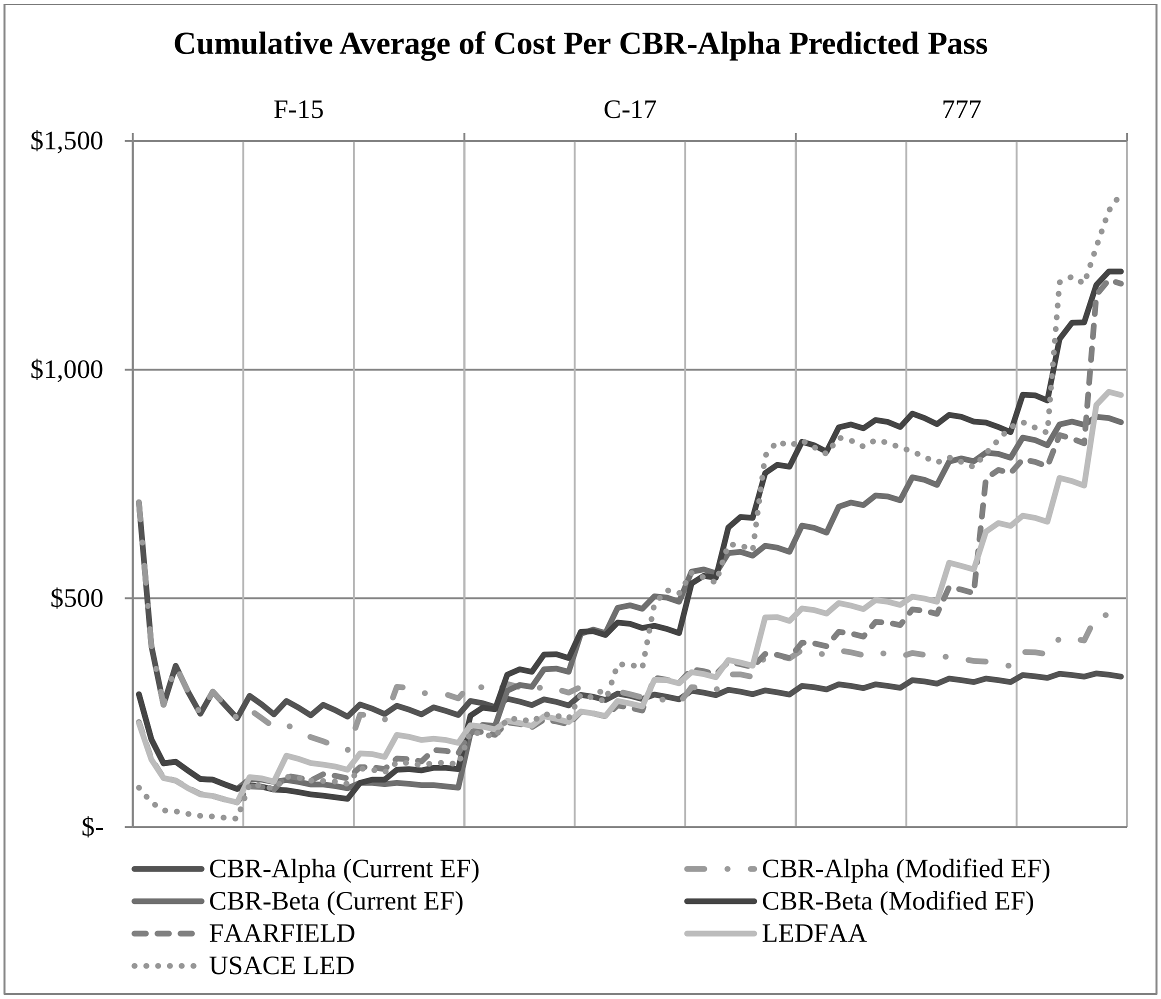


Figure 73. Comparison of the Cumulative Averages of the Cost per CBR-Alpha Predicted Pass Based on the Predicted Thickness Calculated by the Various Flexible Pavement Design Methods – Aggregated Results (n = 81)

Appendix C. Cost Comparison of CBR-Beta Design Method Using Equivalency Factors Based Upon Variable Degrees of Uncertainty

This appendix analyzed the sensitivity of the analysis completed in the previous appendix based upon varying the uncertainty of the equivalency factor. To vary the uncertainty, the research team recorded the percentiles of the calculated equivalency factors from the simulation (Chapter III) at five percent increments. Using the percentiles, the equivalency factors were used to calculate the average difference between using stabilized and conventional base courses, the haul distance required to offset the distance, and the average cost-per-pass of the stabilized pavement.

Cement-Stabilized Base Course Equivalency Factor

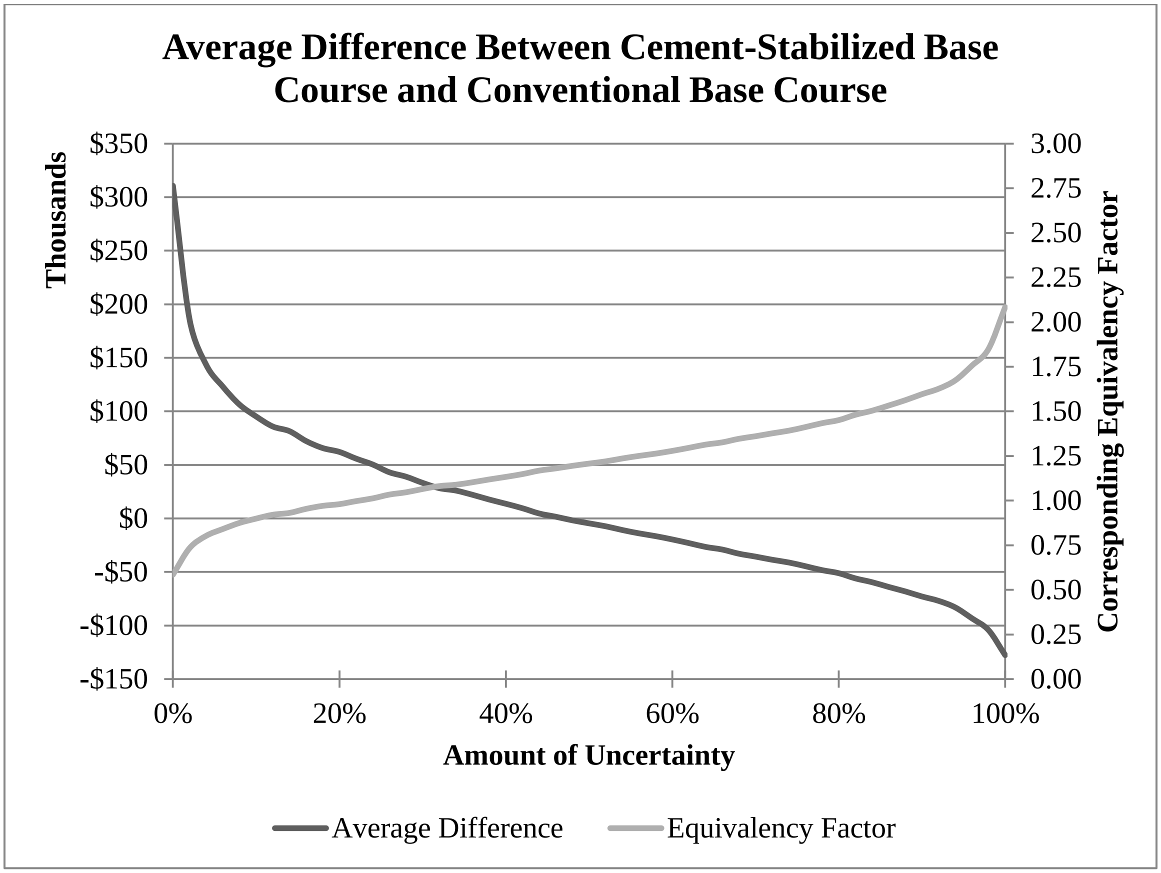


Figure 74. Average Difference in Cost Between Using Cement-Stabilized Base Courses in Lieu of Conventional Base Courses with Varying Degrees of Uncertainty to the Equivalency Factor Derivation

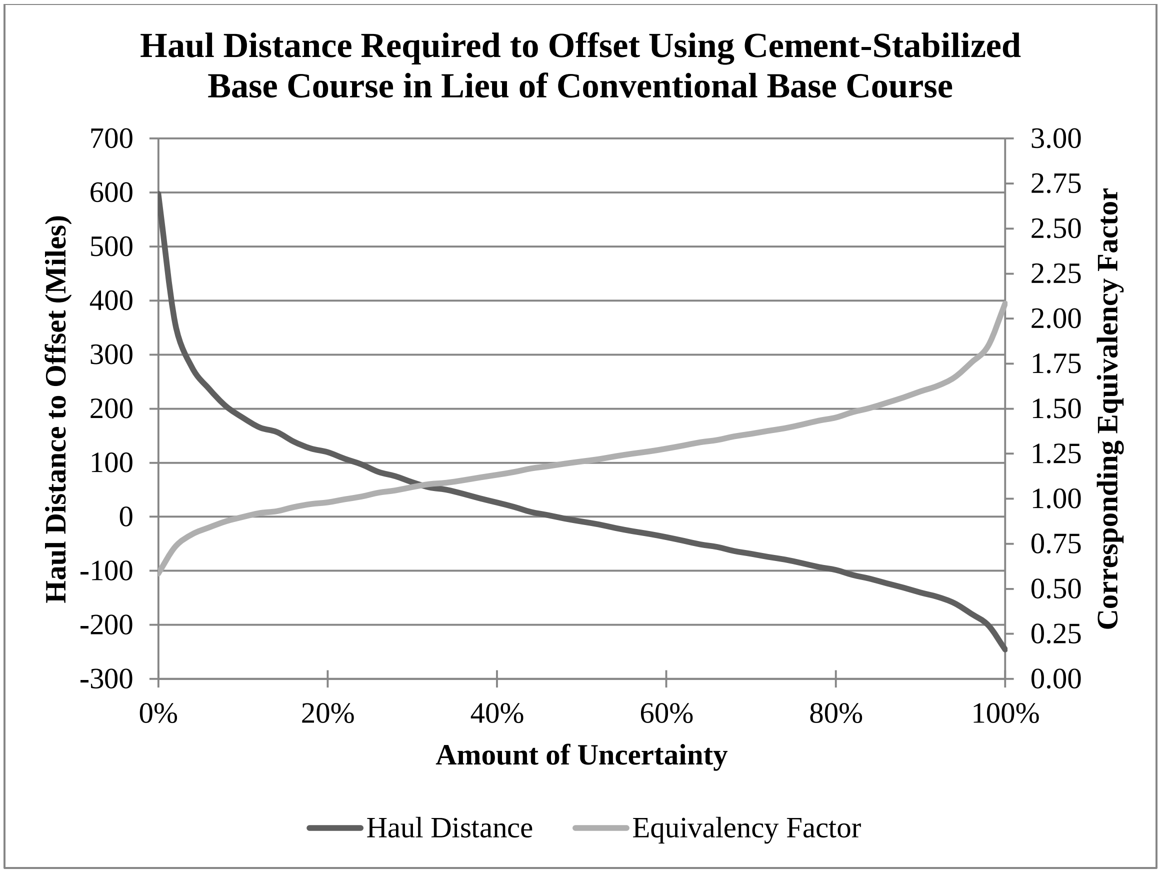


Figure 75. Average Haul Distance to Offset Using Cement-Stabilized Base Course in Lieu of Conventional Base Course with Varying Degrees of Uncertainty to the Equivalency Factor Derivation

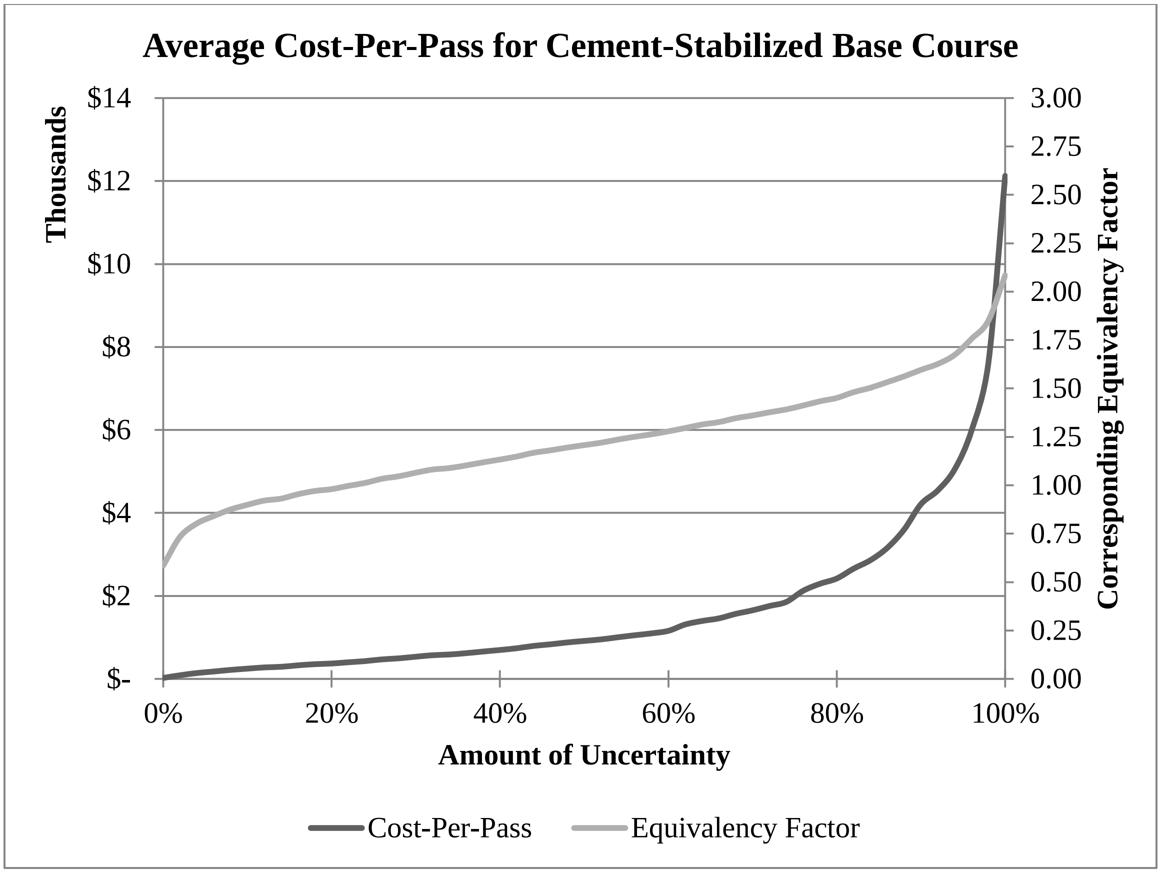


Figure 76. Average Cost-Per-Pass for Cement-Stabilized Base Courses with Varying Degrees of Uncertainty to the Equivalency Factor Derivation

Asphalt-Stabilized Base Course Equivalency Factor

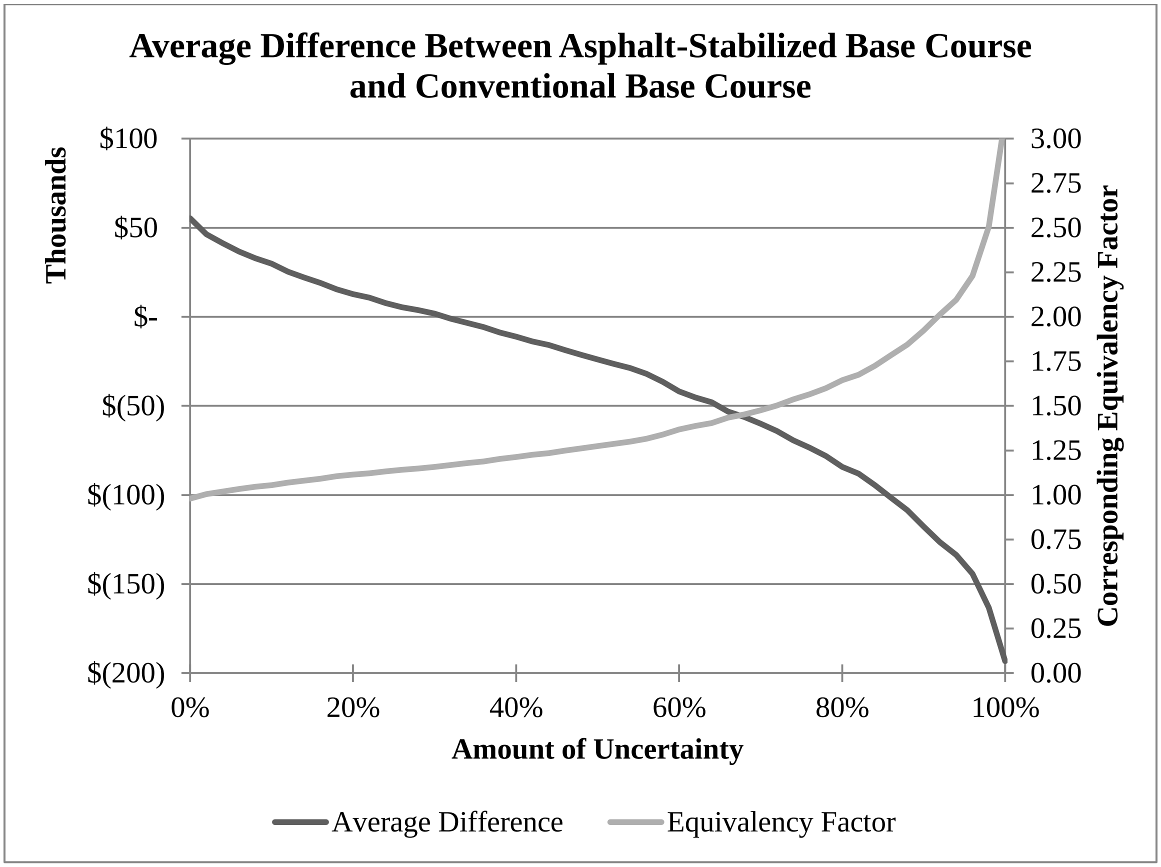


Figure 77. Average Difference in Cost Between Using Asphalt-Stabilized Base Courses in Lieu of Conventional Base Courses with Varying Degrees of Uncertainty to the Equivalency Factor Derivation

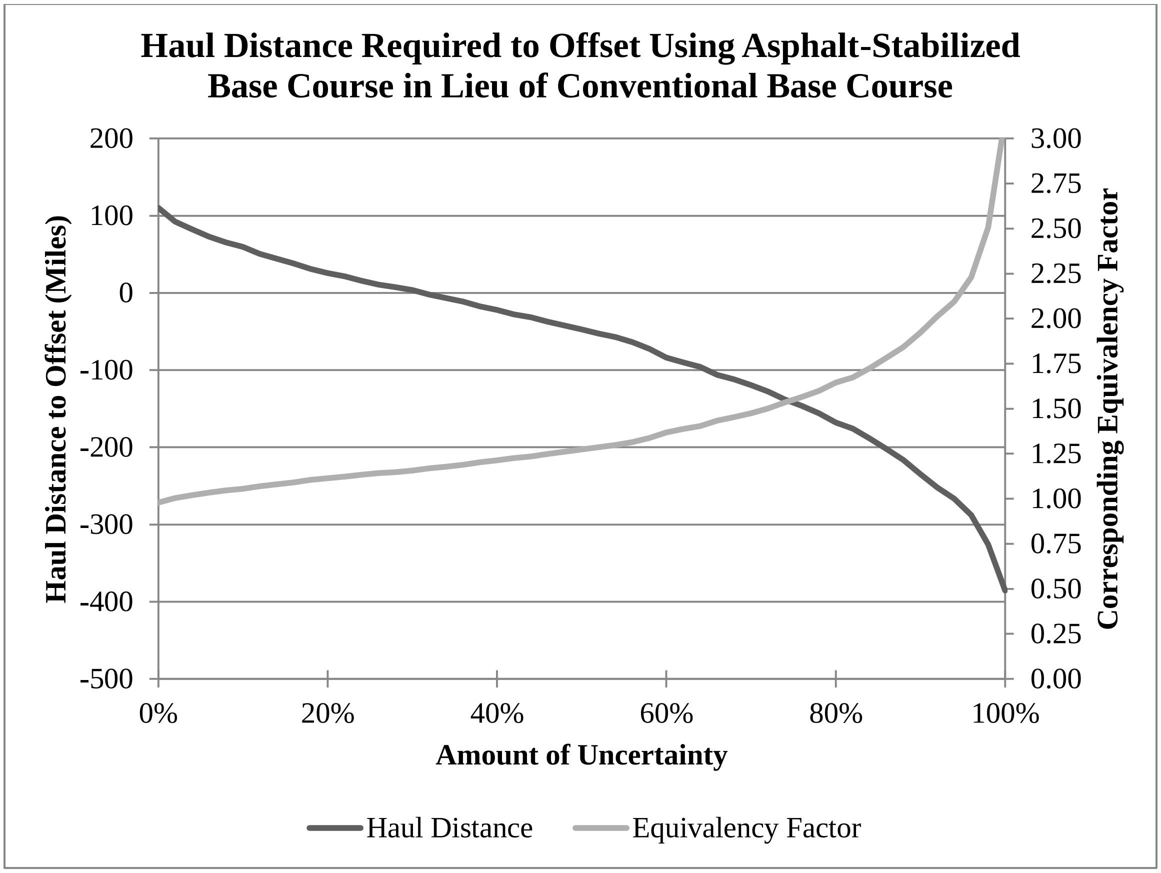


Figure 78. Average Haul Distance to Offset Using Asphalt-Stabilized Base Course in Lieu of Conventional Base Course with Varying Degrees of Uncertainty to the Equivalency Factor Derivation

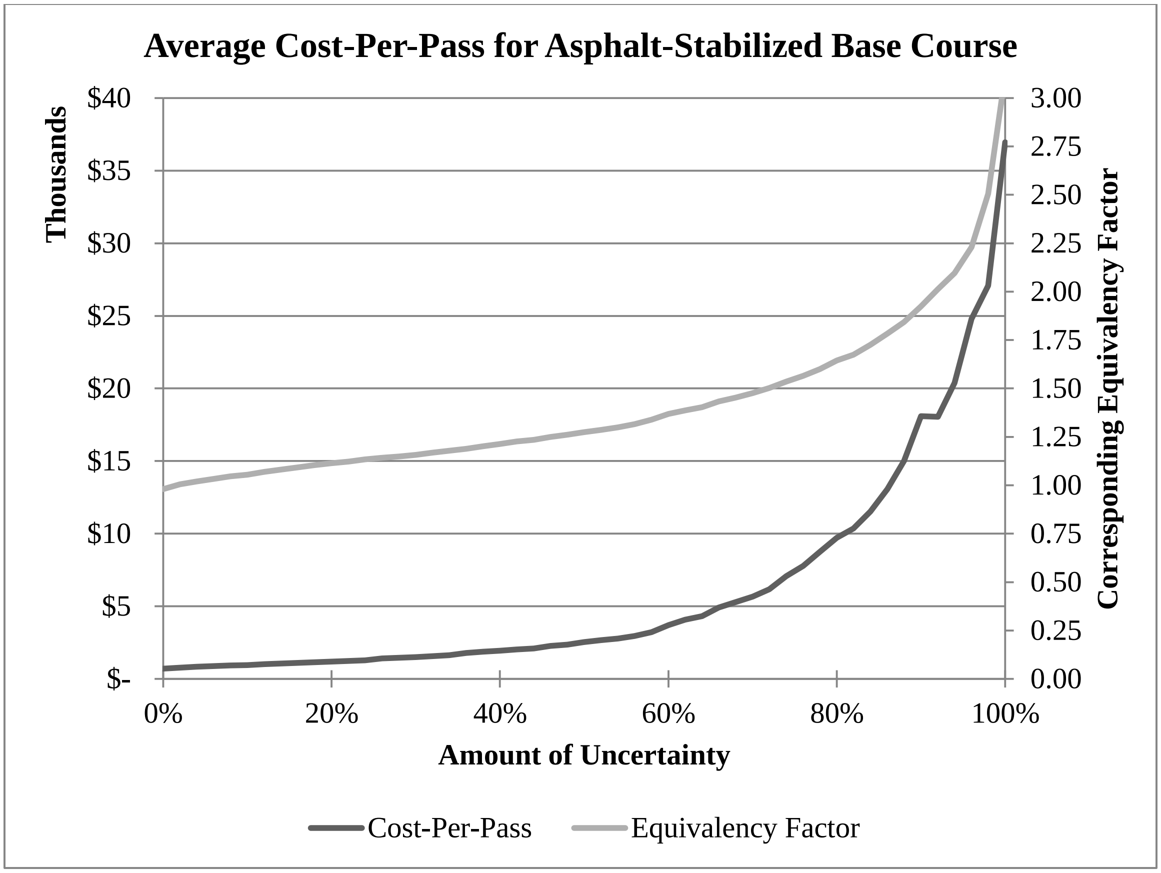


Figure 79. Average Cost-Per-Pass for Asphalt-Stabilized Base Courses with Varying Degrees of Uncertainty to the Equivalency Factor Derivation

Appendix D. Analysis of the Required Line-Haul Distance for the Various Flexible Design Methodologies to Justify Utilizing Soil Stabilization Techniques in Lieu of Hauling Conventional Soils

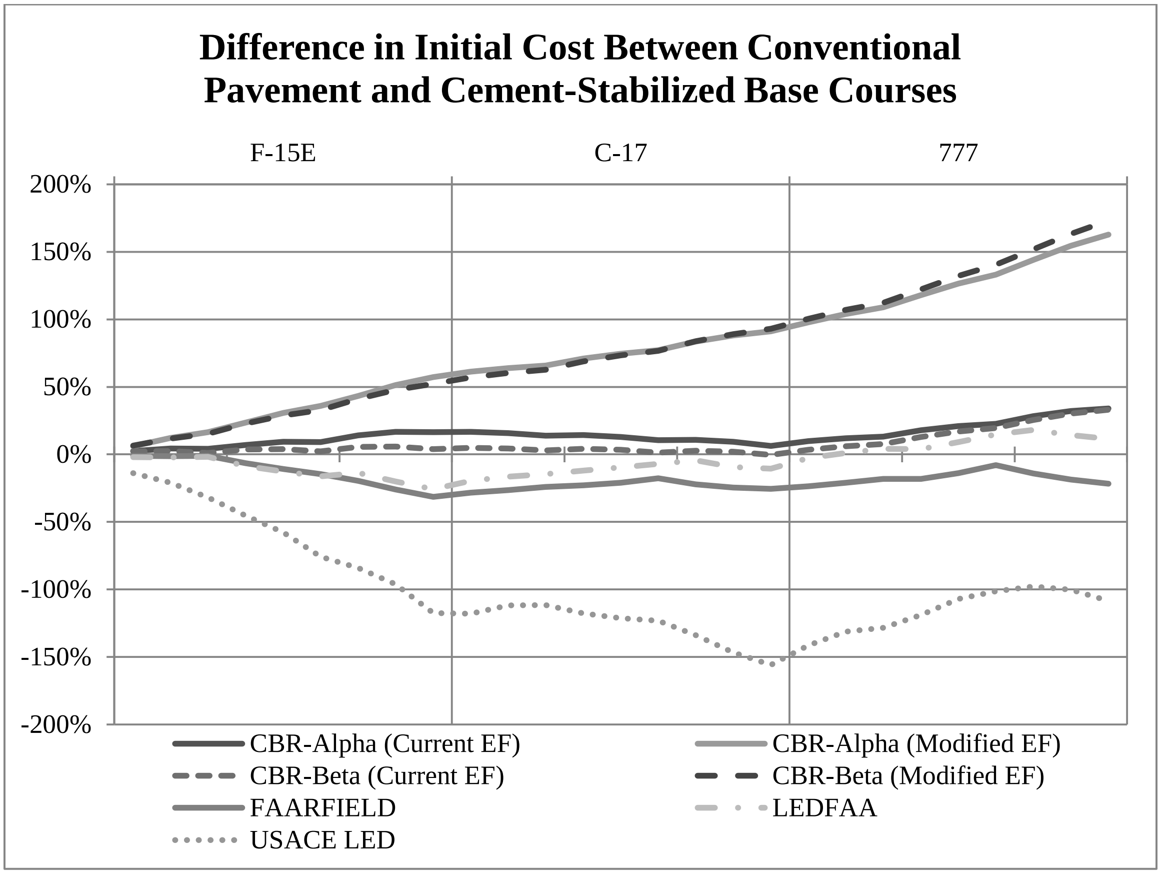


Figure 80. Comparison of the Cumulative Differences in the Relative Difference Between the Initial Construction Cost of Cement-Stabilized Base Courses and Conventional Base Courses

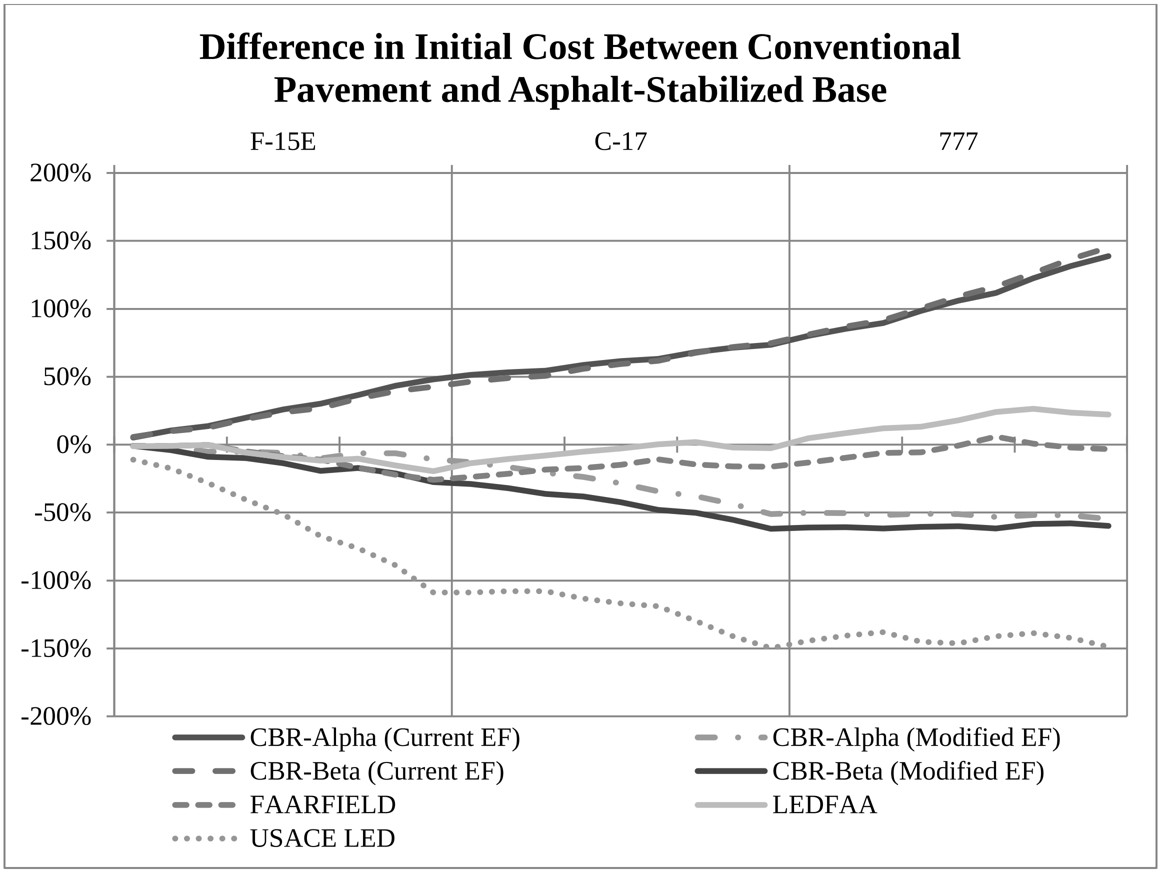
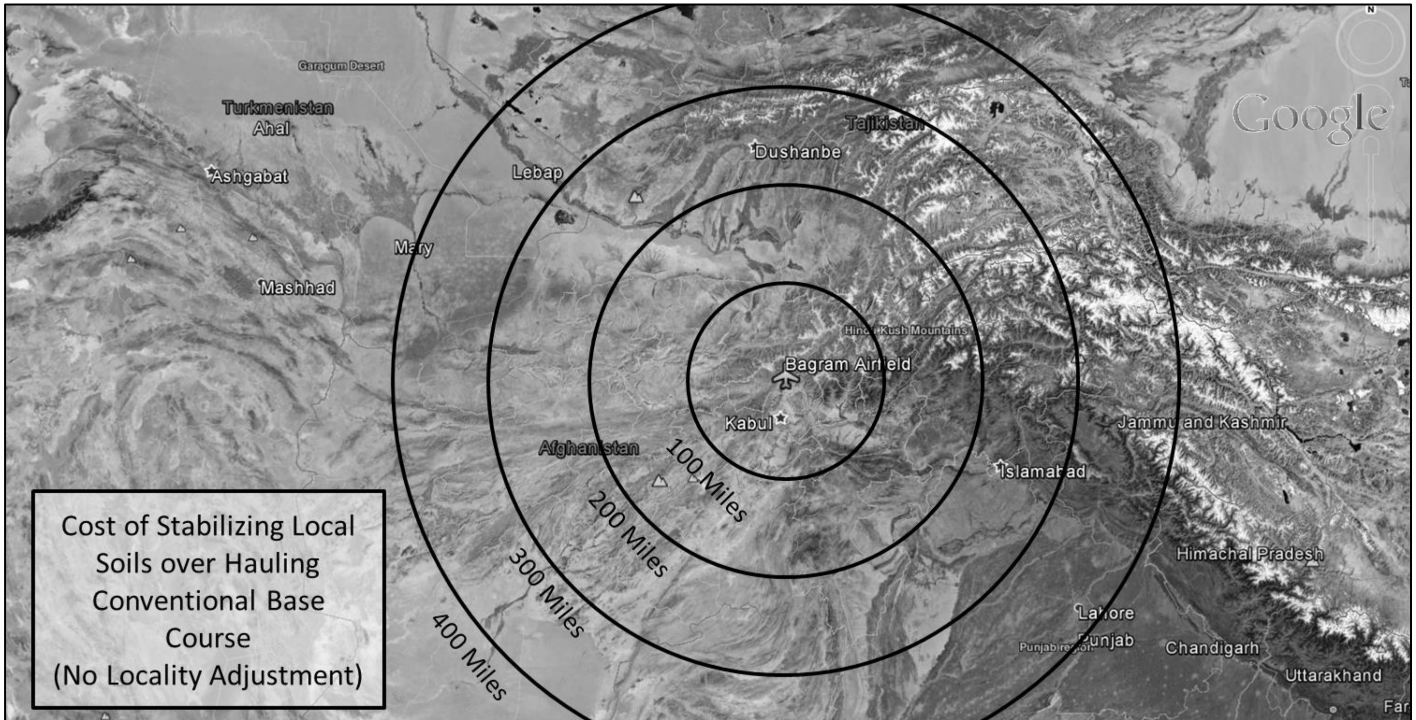


Figure 81. Comparison of the Cumulative Differences in the Relative Difference Between the Initial Construction Cost of Asphalt-Stabilized Base Courses and Conventional Base Courses



		CBR-Alpha (Current EF)		CBR-Alpha (Modified EF)		CBR-Beta (Current EF)		CBR-Beta (Modified EF)		FAARFIELD		LEDFAA		USACE LED	
		Average Difference	Haul Distance to Offset (Miles)	Average Difference	Haul Distance to Offset (Miles)	Average Difference	Haul Distance to Offset (Miles)	Average Difference	Haul Distance to Offset (Miles)	Average Difference	Haul Distance to Offset (Miles)	Average Difference	Haul Distance to Offset (Miles)	Average Difference	Haul Distance to Offset (Miles)
Cement-Stabilized Base (GW, GP, GM GC) over Conventional Subbase	F-15	$11,725	31.56	$44,303	119.24	$2,167	5.24	$41,433	100.16	-$24,318	-38.90	-$20,474	-33.31	-$109,134	-219.53
	C-17	-$11,883	-21.71	$35,099	64.14	-$5,079	-10.39	$38,741	79.25	$10,596	15.63	$19,429	34.59	-$29,620	-59.89
	777	$33,562	52.77	$89,206	140.25	$37,527	62.36	$91,076	151.34	$12,778	16.02	$32,483	44.48	$70,065	122.77
	Overall	$11,135	21.48	$56,203	108.44	$11,538	23.01	$57,083	113.84	-$315	-0.45	$10,480	16.49	-$22,897	-43.96
Asphalt-Stabilized Base (GW, GP, GM, GC) over Conventional Subbase	F-15	$37,116	99.89	-$10,175	-27.38	$33,568	81.14	-$23,577	-56.99	-$19,647	-31.43	-$15,249	-24.81	-$4,706	-9.47
	C-17	$26,006	47.52	-$44,064	-80.52	$30,237	61.85	-$34,559	-70.69	$14,515	21.41	$21,591	38.44	-$5,409	-10.94
	777	$80,799	127.04	-$5,971	-9.39	$80,136	133.16	$1,183	1.97	$22,765	28.53	$35,754	48.96	-$58,367	-102.27
	Overall	$47,974	92.56	-$20,070	-38.72	$47,980	95.68	-$18,984	-37.86	$5,877	8.39	$14,032	22.08	-$22,828	-43.83

Figure 82. Comparison of the Costs of Using Stabilized Local Soils over Hauling Conventional Base Course

Appendix E. Comparison of the Various Layered-Elastic Design Programs Using the PCASE Evaluation Module

Table 22. Comparison of the Layered-Elastic Design Programs for an F-15E at 20,000 Passes with Varying Subgrade CBR Values: Design Outputs

				Wearing Course			Base Course			Subbase Course (1)			Subbase Course (2)			Subgrade	
Structure	Program	Subgrade CBR	Passes	Thickness	Modulus	ν	Thickness	Modulus	ν	Thickness	Modulus	ν	Thickness	Modulus	ν	Modulus	ν
Conventional Base over Conventional Subbase	PSEVEN	3	20,000	5	200,000	0.50	8.79	46,496	0.30	8	18,978	0.30	7.68	10,399	0.3	4,500	0.40
		6	20,000	5	200,000	0.50	8.79	45,447	0.30	10.59	18,328	0.30				9,000	0.40
		10	20,000	5	200,000	0.50	8.79	51,766	0.30	5.59	22,462	0.30				15,000	0.40
		15	20,000	5	200,000	0.50	10.87	54,680	0.30							22,500	0.40
		20	20,000	5	200,000	0.50	8.79	61,670	0.30							30,000	0.40
	FAARFIELD	3	20,000	4	200,000	0.35	12.48	62,848	0.35	17.8	16,966	0.35				4,500	0.35
		6	20,000	4	200,000	0.35	12.48	60,640	0.35	10.42	19,098	0.35				9,000	0.35
		10	20,000	4	200,000	0.35	12.48	60,442	0.35	5.66	22,512	0.35				15,000	0.35
		15	20,000	4	200,000	0.35	14.55	63,403	0.35							22,500	0.35
		20	20,000	4	200,000	0.35	12.48	68,779	0.35							30,000	0.35
	LEDFAA	3	20,000	4	200,000	0.35	12.29	62,706	0.35	18.01	17,083	0.35				4,500	0.35
		6	20,000	4	200,000	0.35	12.29	60,492	0.35	10.58	19,188	0.35				9,000	0.35
		10	20,000	4	200,000	0.35	12.29	60,241	0.35	5.76	22,592	0.35				15,000	0.35
		15	20,000	4	200,000	0.35	14.41	63,208	0.35							22,500	0.35
		20	20,000	4	200,000	0.35	12.29	68,504	0.35							30,000	0.35
	PCASE LED	3	20,000	5	200,000	0.50	9.7	51,673	0.30	11	21,505	0.30	10.85	11,422	0.3	4,500	0.40
		6	20,000	5	200,000	0.50	9.7	57,380	0.30	11.78	18,750	0.30				9,000	0.40
		10	20,000	5	200,000	0.50	9.7	53,940	0.30	6.4	23,045	0.30				15,000	0.40
		15	20,000	5	200,000	0.50	12.05	56,075	0.30							22,500	0.40
		20	20,000	5	200,000	0.50	9.7	63,098	0.30							30,000	0.40

Table 23. Comparison of the Design Outputs for the Various Layered-Elastic Design Programs for an F-15E at 20,000 Passes and Varying Subgrade CBR Values

				PCASE Evaluation LED			PCASE Evaluation CBR			CBR-Beta Passes	
Structure	Program	Subgrade CBR	Passes	ACN/PCN	Allowable Load (kips)	Allowable Passes	ACN/PCN	Allowable Load (kips)	Allowable Passes	Equivalent Thickness	Allowable Passes
Conventional Base over Conventional Subbase	PSEVEN	3	20,000	1.5	52.3	842	1.1	70.1	4,821	29.47	5,058
		6	20,000	1.3	63.3	1,695	0.9	84.8	30,539	24.38	45,876
		10	20,000	1.2	66.2	1,520	0.9	84.8	30,539	19.38	43,949
		15	20,000	1.2	67.9	1,336	0.9	84.3	29,900	15.87	36,339
		20	20,000	1.2	69.1	1,237	0.9	84.8	30,539	13.79	38,942
	FAARFIELD	3	20,000	0.7	113	105,593	1.2	68.7	13,657	32.98	52,685
		6	20,000	1.0	80.3	18,121	1.2	68.7	13,657	25.60	150,886
		10	20,000	0.9	83.7	32,135	1.2	68.7	13,657	20.84	251,845
		15	20,000	0.9	87.3	75,742	1.2	68.7	13,657	17.25	238,385
		20	20,000	0.9	89.4	133,175	1.2	68.7	13,657	15.18	319,209
	LEDFAA	3	20,000	1.1	75.3	11,735	1.2	68.7	13,657	33.00	53,456
		6	20,000	1.0	79.8	17,150	1.2	68.7	13,657	25.57	146,357
		10	20,000	1.0	83	28,116	1.2	68.7	13,657	20.75	224,714
		15	20,000	0.9	86.1	59,453	1.2	68.7	13,657	17.11	194,617
		20	20,000	0.9	87.6	104,271	1.2	68.7	13,657	14.99	234,712
	PCASE LED	3	20,000	1.0	77.7	14,767	0.8	96.2	45,759	36.55	907,608
		6	20,000	1.0	77	11,632	0.8	96.2	45,759	26.48	379,804
		10	20,000	1.0	76.6	8,993	0.8	96.2	45,759	21.10	351,841
		15	20,000	0.9	93.1	62,100	0.8	97.4	45,759	17.05	178,569
		20	20,000	1.1	76.2	5,570	0.8	96.2	45,759	14.70	148,682

Appendix F. Supporting Documentation for Effect of Stabilized Layer Thickness and Subgrade Depth on the Concentration Factor for Flexible Airfield Pavements Using the CBR-Beta Design Method

The results presented herein document the statistical evaluation of the status quo, best performing single-layer, and the best performing two-layer concentration factor models. Documentation for these models include a plot of the equivalent compared to predicted thicknesses, a histogram of relative error (Δ), and a histogram of the absolute percentage error scores. Additionally, after the results for each model are presented, similar graphs are included for the aggregated results of the three models combined.

Comparison of Equivalent and Predicted Thicknesses of Individual Models

Status Quo Concentration Factor Model

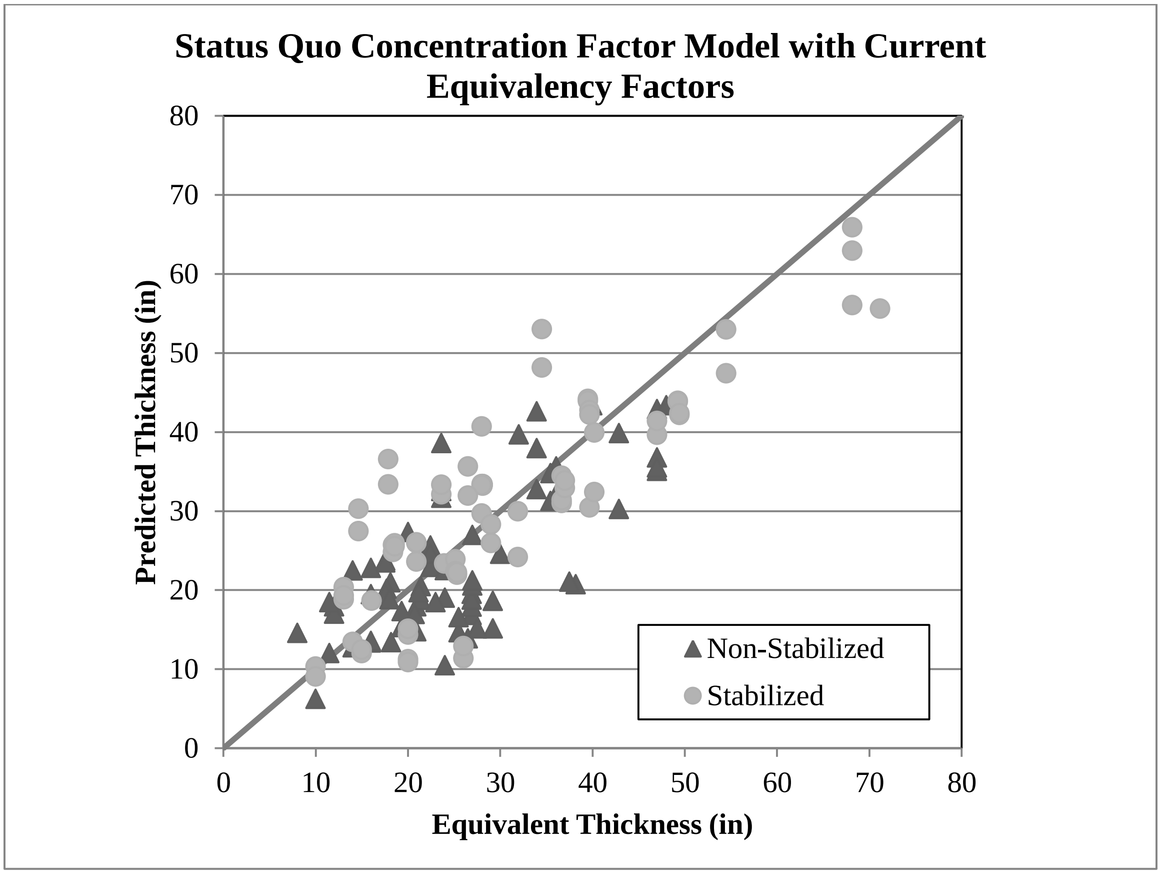


Figure 83. Comparison of Equivalent and Predicted Thickness for the Status Quo Concentration Factor Model with Current Equivalency Factors (n = 157)

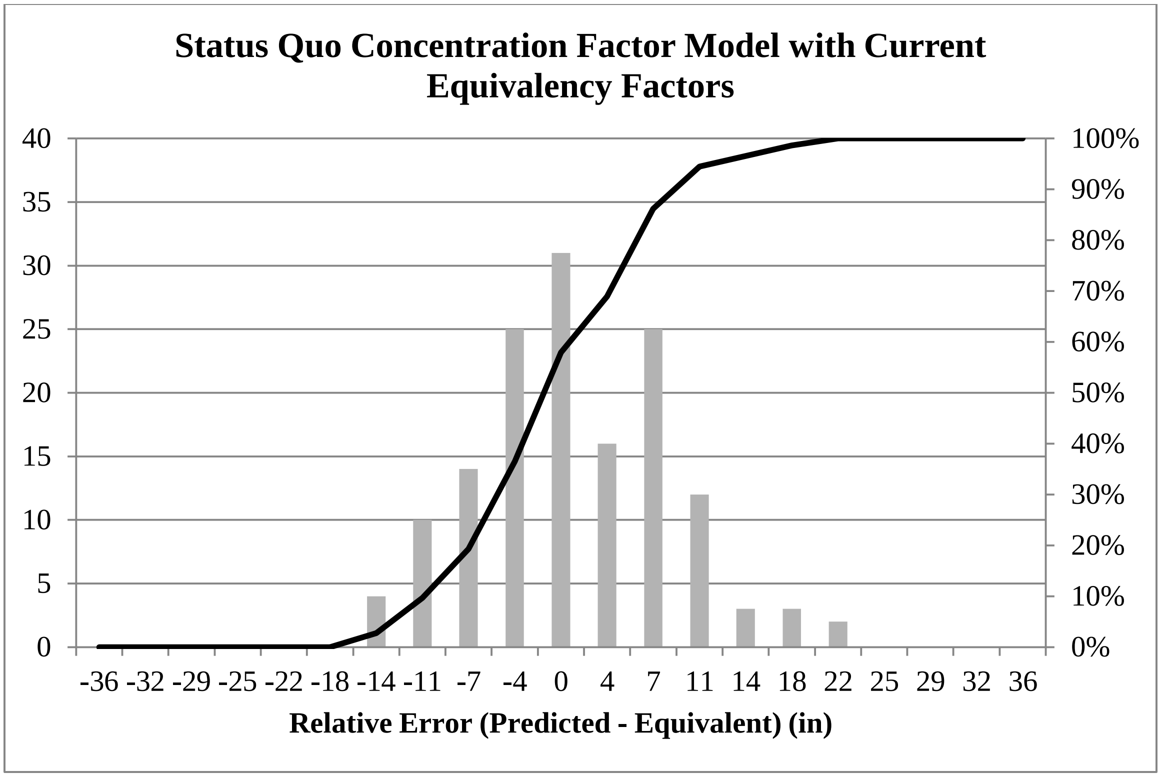


Figure 84. Histogram of the Relative Error Between the Predicted and Equivalent Thicknesses for the Status Quo Concentration Factor Model with Current Equivalency Factors (n = 157)

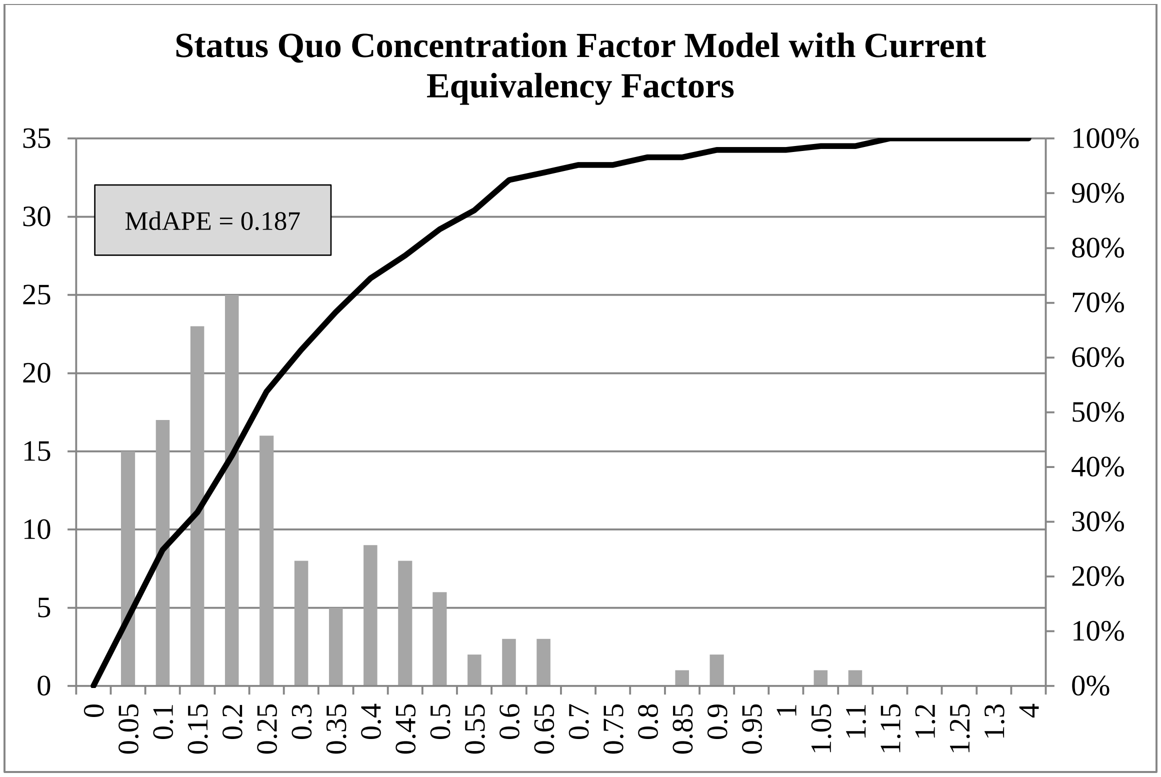


Figure 85. Histogram of the Absolute Percentage Error Between the Predicted and Equivalent Thicknesses for the Status Quo Concentration Factor Model with Current Equivalency Factors (n = 157)

__Single-Layer Concentration Factor Model__

<u>Model #4</u>

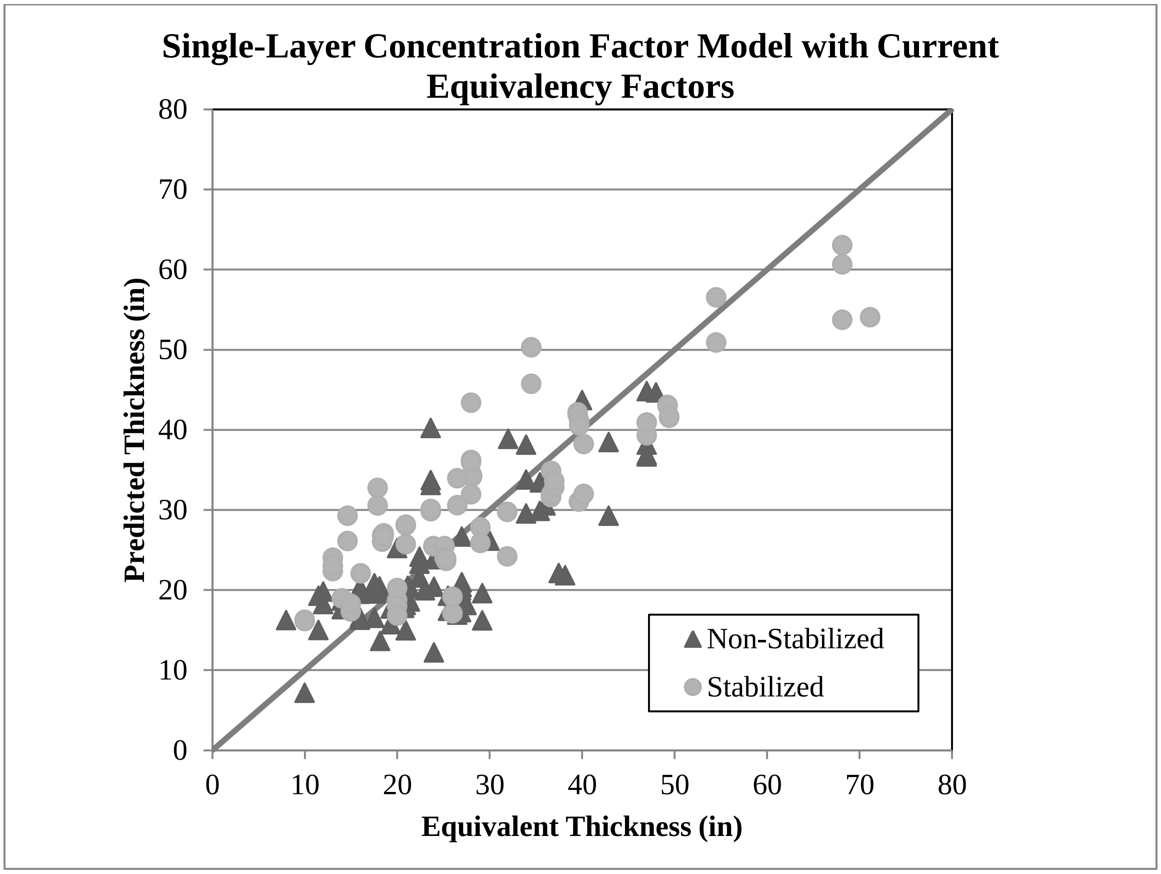


Figure 86. Comparison of Equivalent and Predicted Thickness for the Single-Layer Concentration Factor Model with Current Equivalency Factors (n = 157)

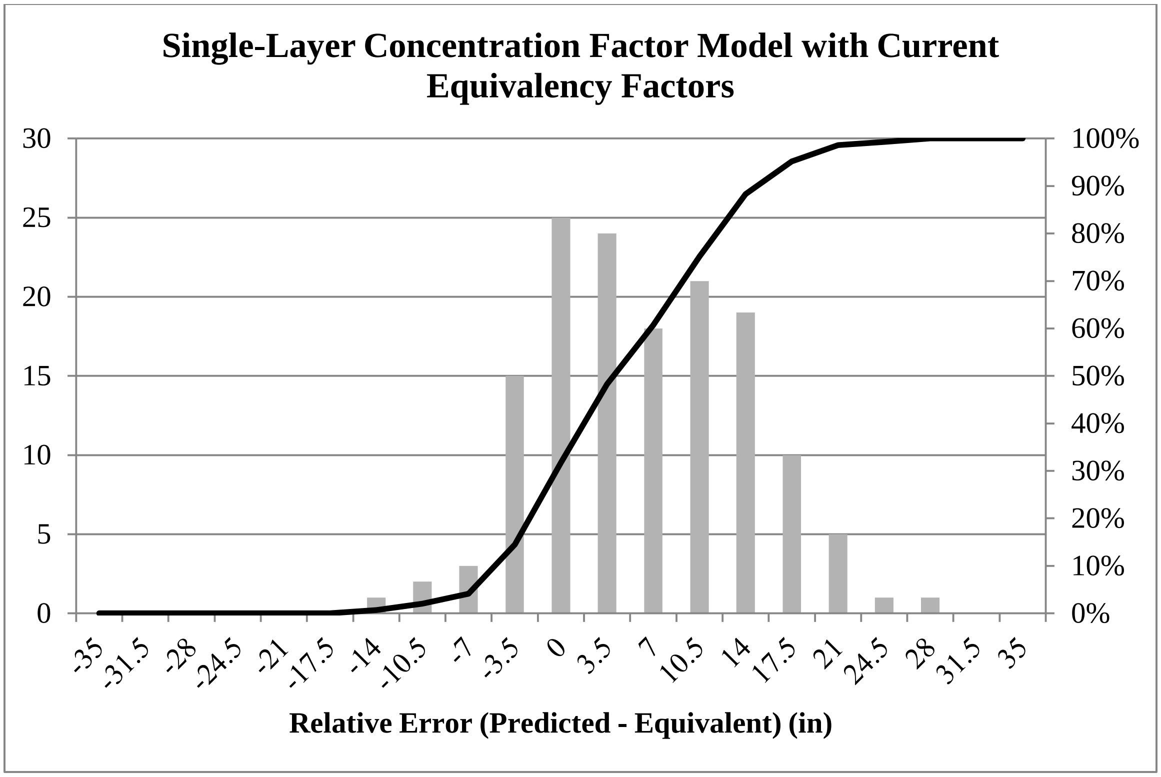


Figure 87. Histogram of the Relative Error Between the Predicted and Equivalent Thicknesses for the Single-Layer Concentration Factor Model with Current Equivalency Factors (n = 157)

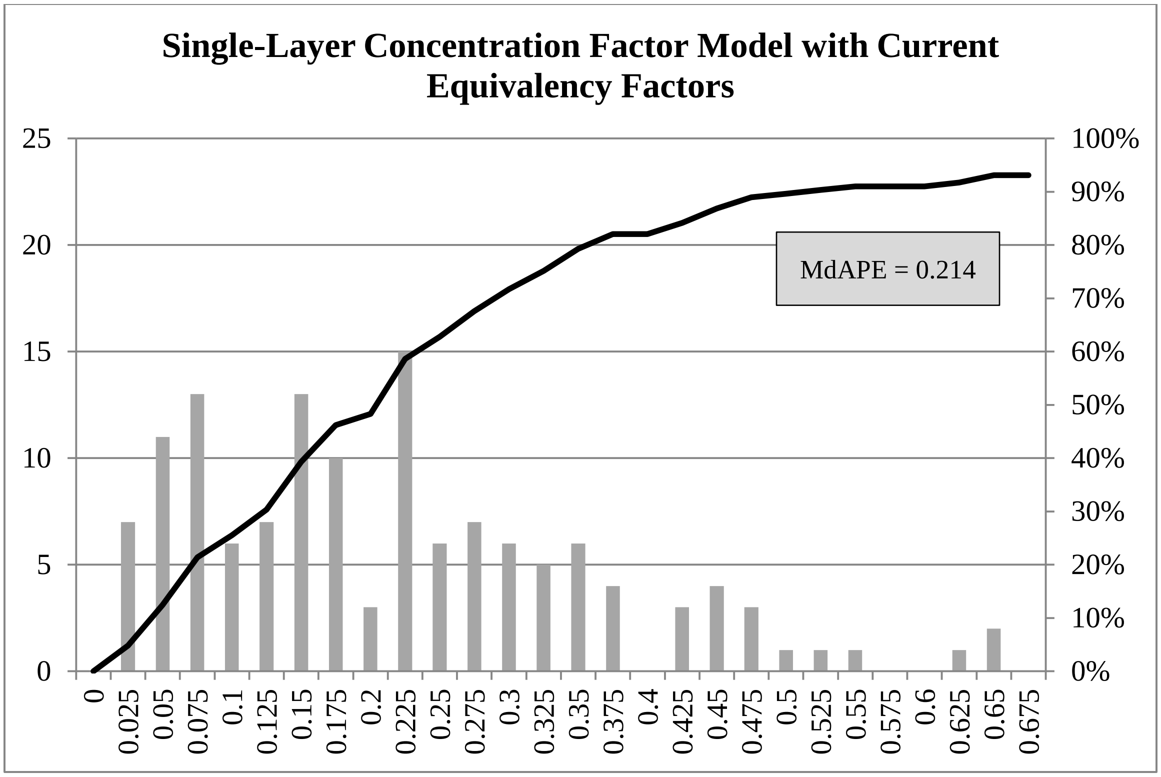


Figure 88. Histogram of the Absolute Percentage Error Between the Predicted and Equivalent Thicknesses for the Single-Layer Concentration Factor Model with Current Equivalency Factors (n = 157)

Two-Layer Concentration Factor Model

<u>Subbase Concentration Factor Model</u>

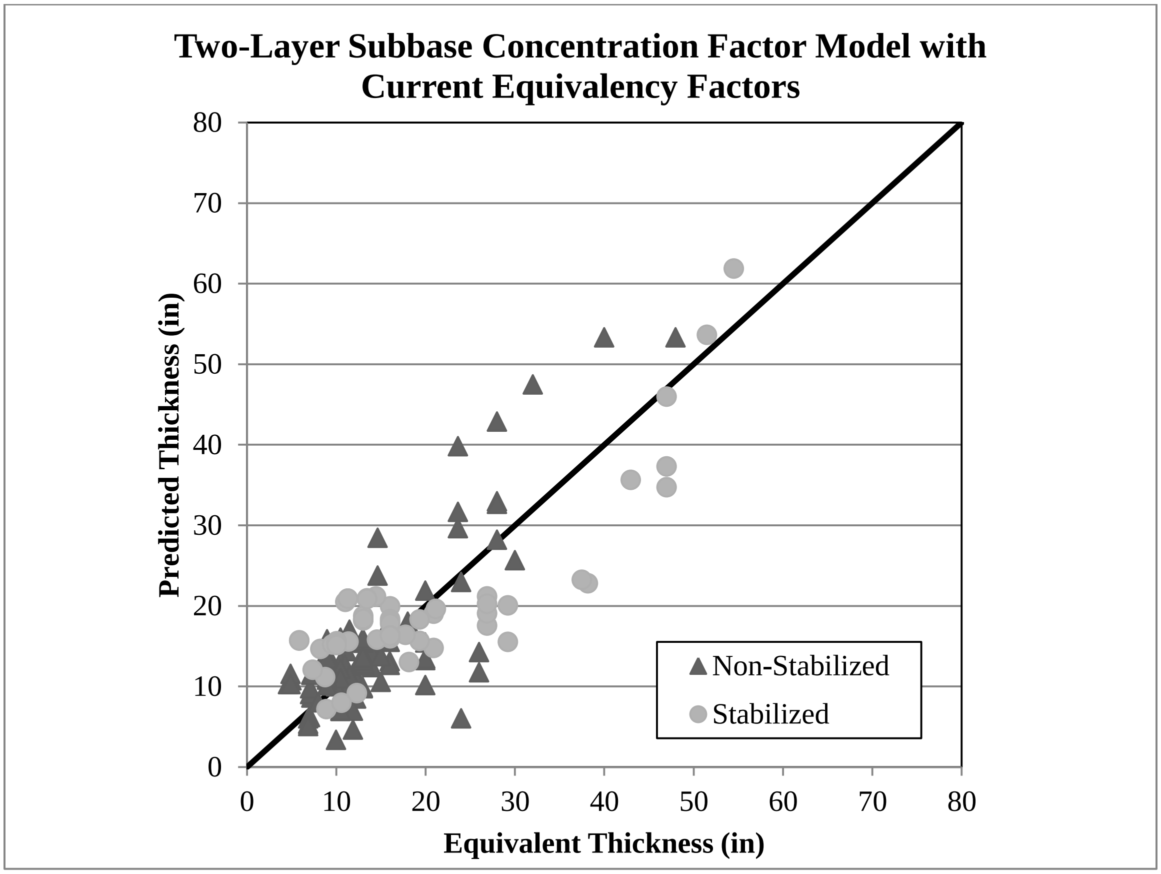


Figure 89. Comparison of Equivalent and Predicted Thickness for the Two-Layer Concentration Factor Model with Current Equivalency Factors for the Subbase (n = 157)

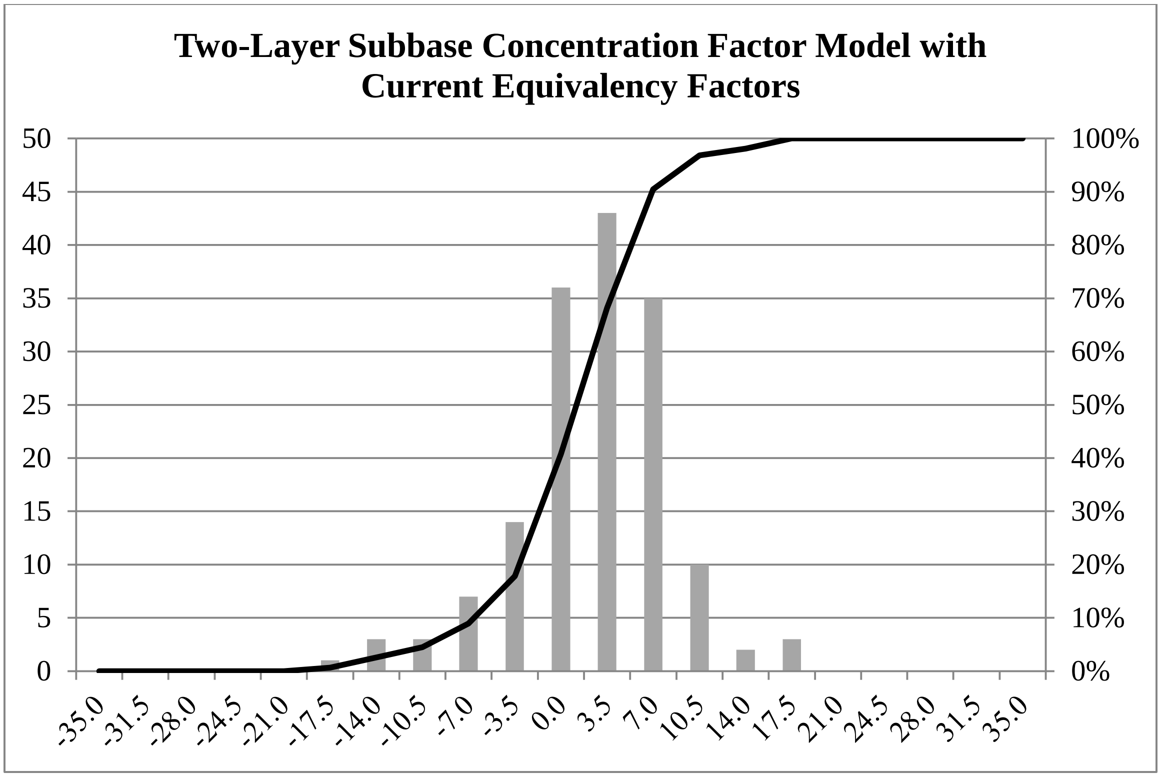


Figure 90. Histogram of the Relative Error Between the Predicted and Equivalent Thicknesses for the Two-Layer Subbase Concentration Factor Model with Current Equivalency Factors (n = 157)

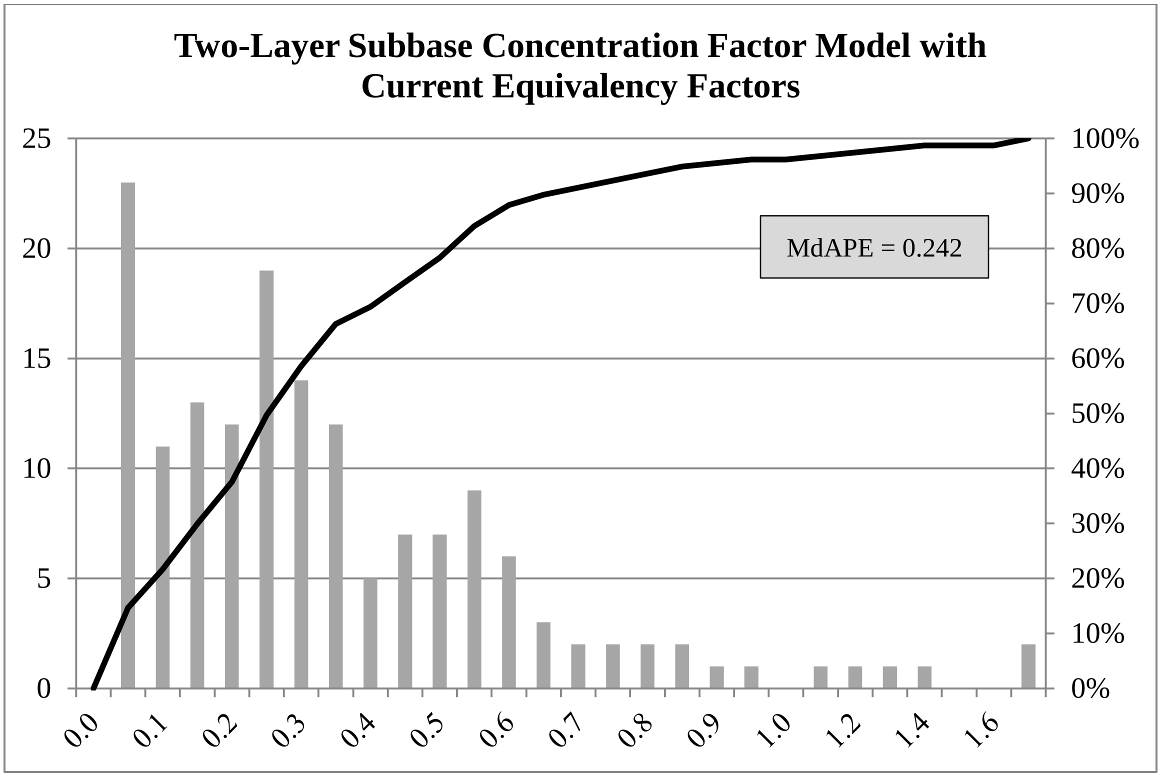


Figure 91. Histogram of the Absolute Percentage Error Between the Predicted and Equivalent Thicknesses for the Two-Layer Subbase Concentration Factor Model with Current Equivalency Factors (n = 157)

Subgrade Concentration Factor Model

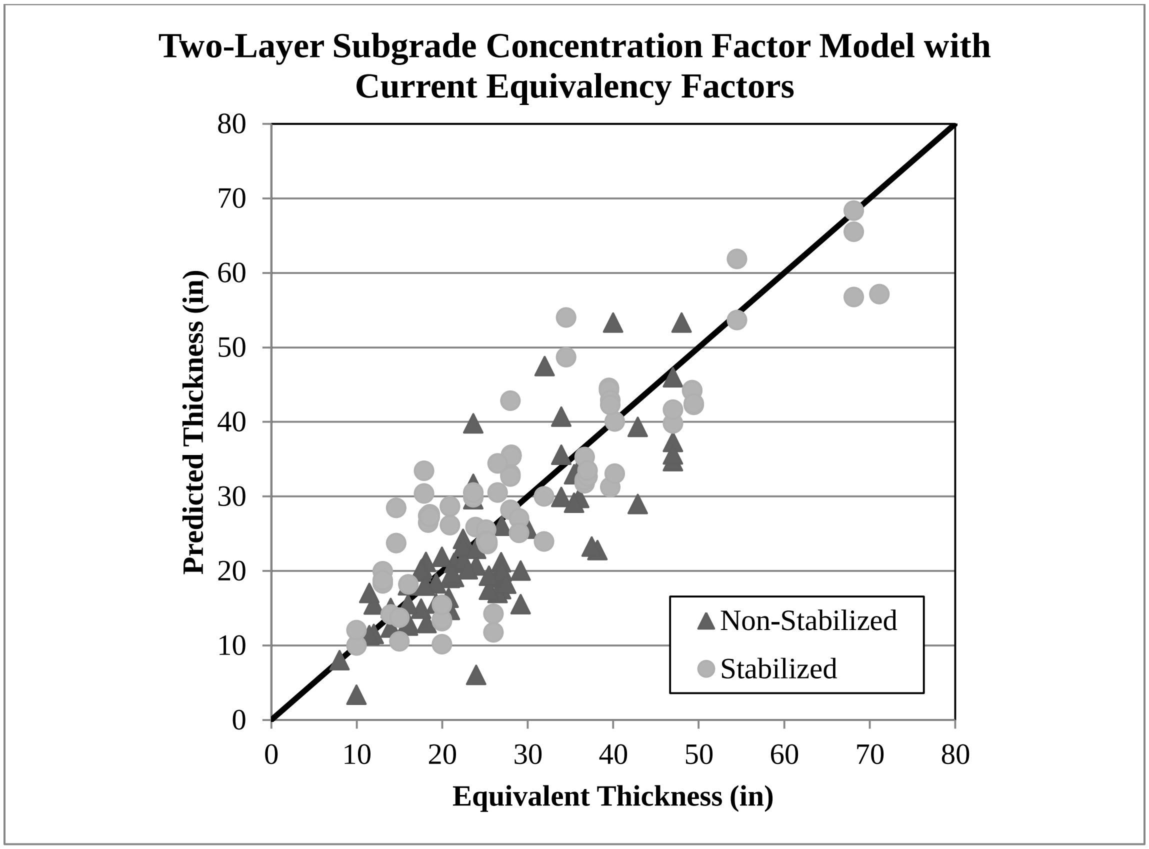


Figure 92. Comparison of Equivalent and Predicted Thickness for the Two-Layer Concentration Factor Model with Current Equivalency Factors for the Subgrade (n = 157)

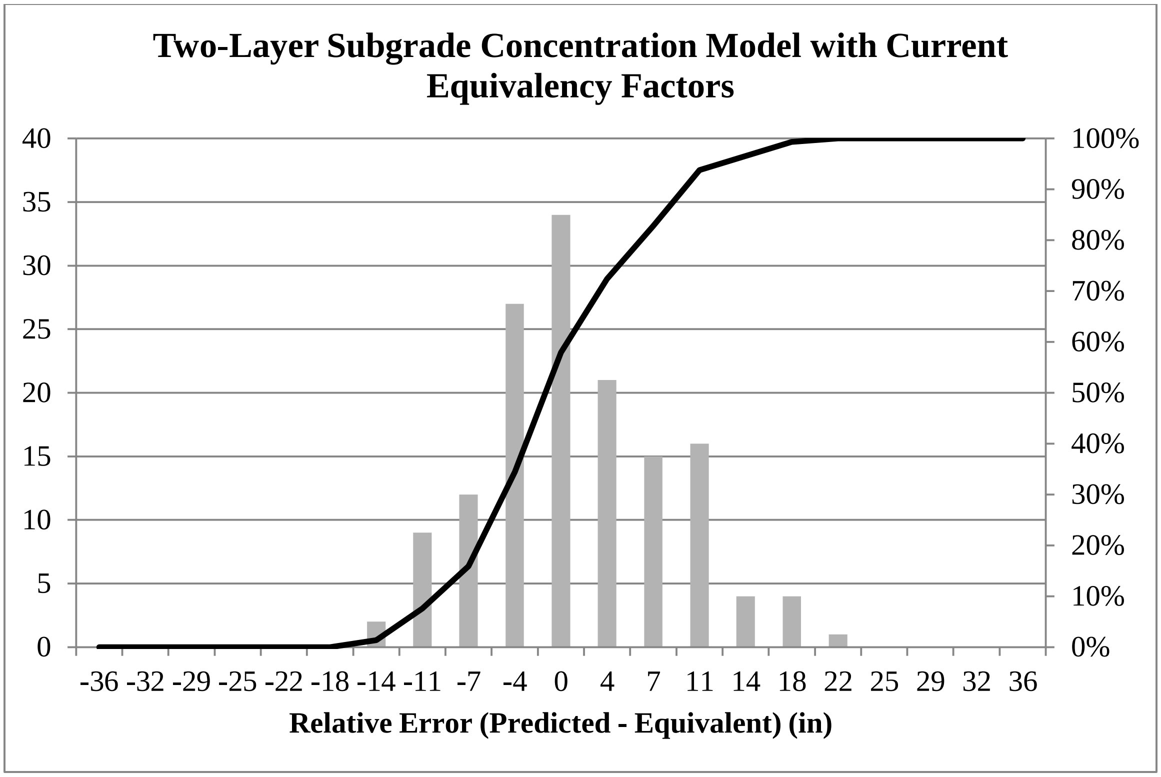


Figure 93. Histogram of the Relative Error Between the Predicted and Equivalent Thicknesses for the Two-Layer Subgrade Concentration Factor Model with Current Equivalency Factors (n = 157)

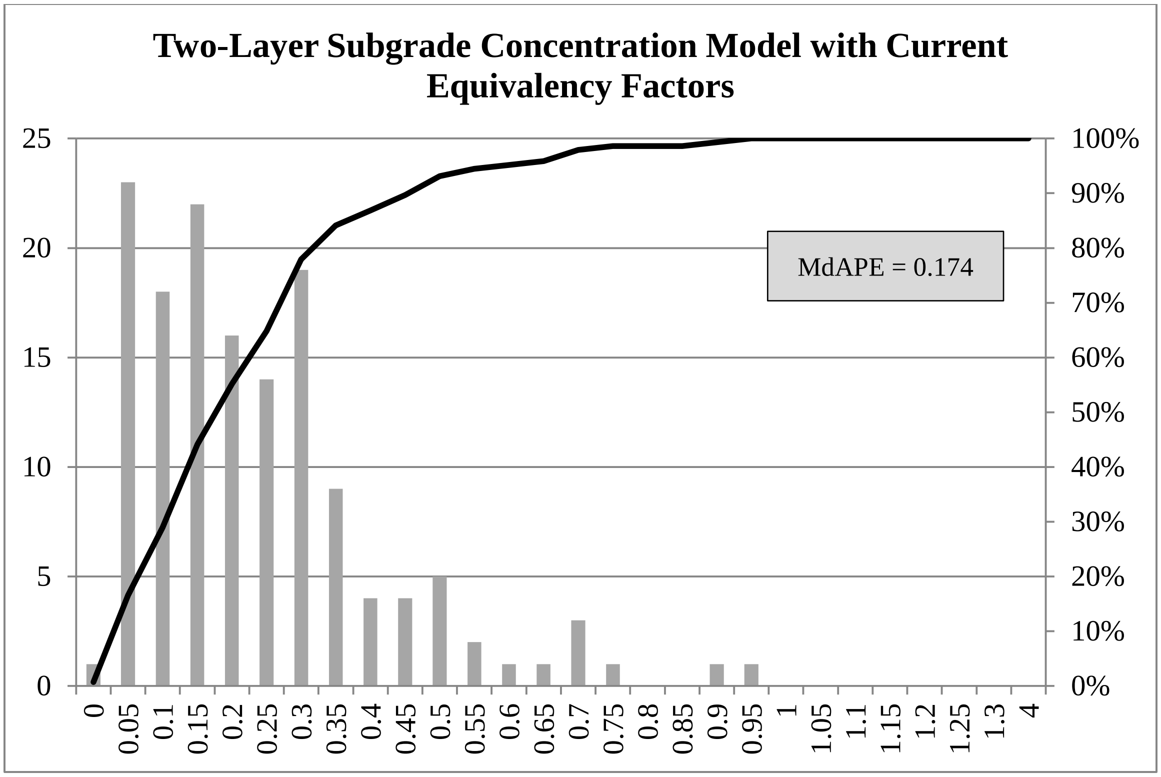


Figure 94. Histogram of the Absolute Percentage Error Between the Predicted and Equivalent Thicknesses for the Two-Layer Subgrade Concentration Factor Model with Current Equivalency Factors (n = 157)

Aggregated Results

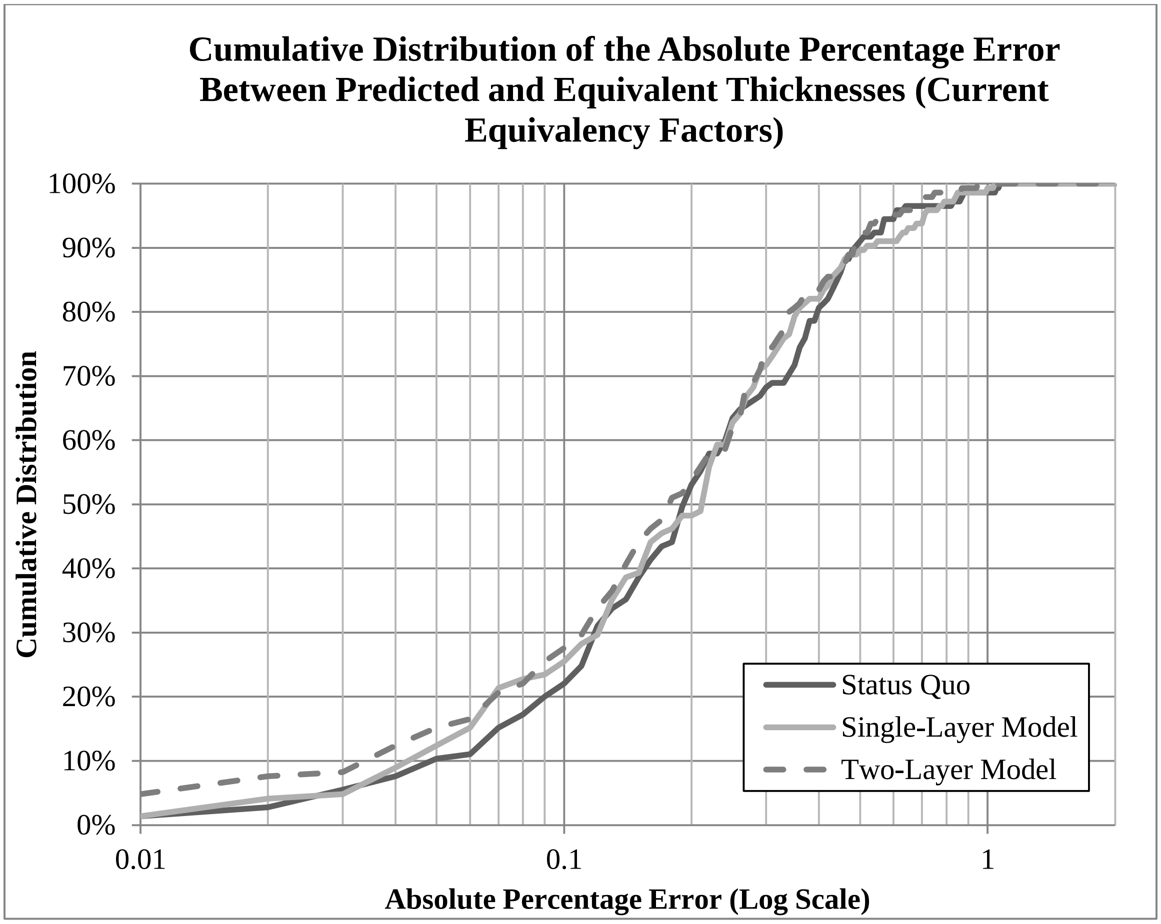


Figure 95. Cumulative Distributions of the Absolute Percentage Error Between the Predicted and Equivalent Thicknesses for the Various Concentration Factor Models with Current Equivalency Factors (n = 157)

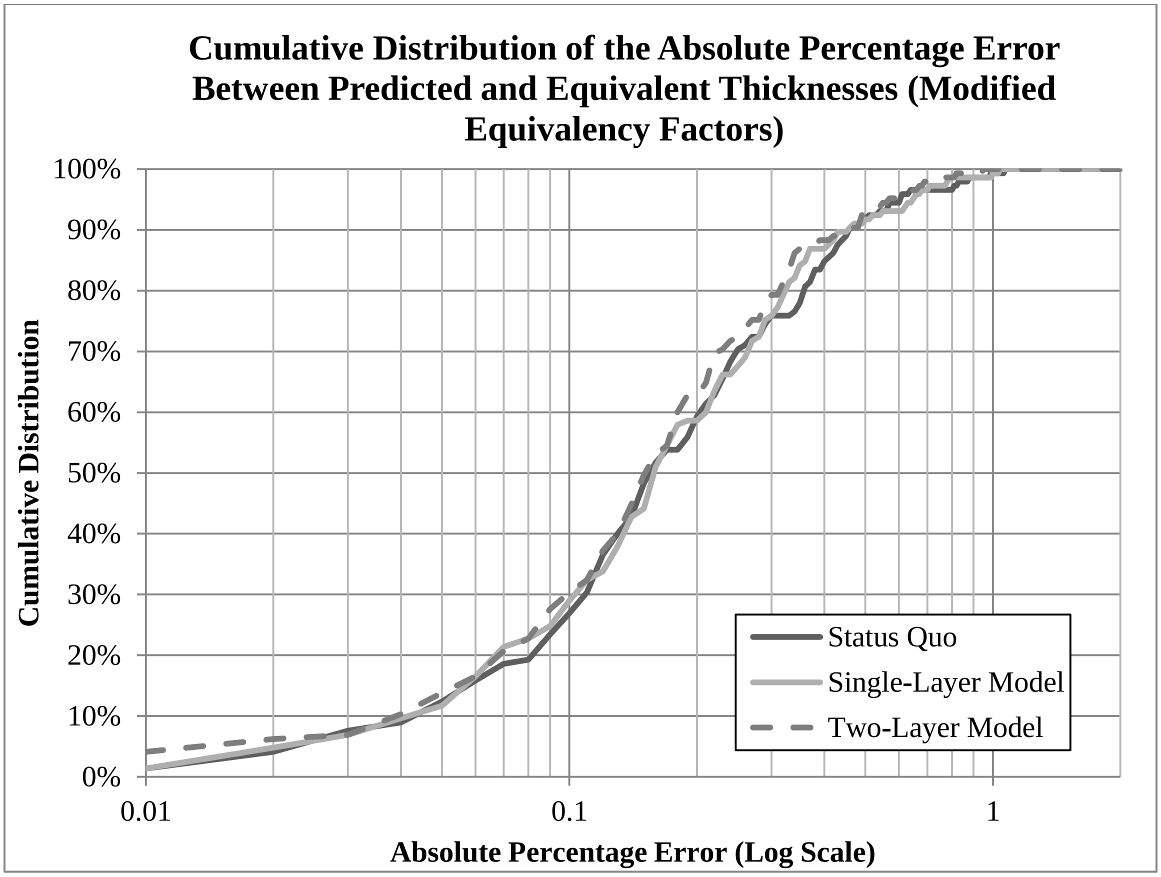


Figure 96. Cumulative Distributions of the Absolute Percentage Error Between the Predicted and Equivalent Thicknesses for the Various Concentration Factor Models with Modified Equivalency Factors (n = 157)

Appendix G. Cost Analysis of the Two-Layer Concentration Factor Model

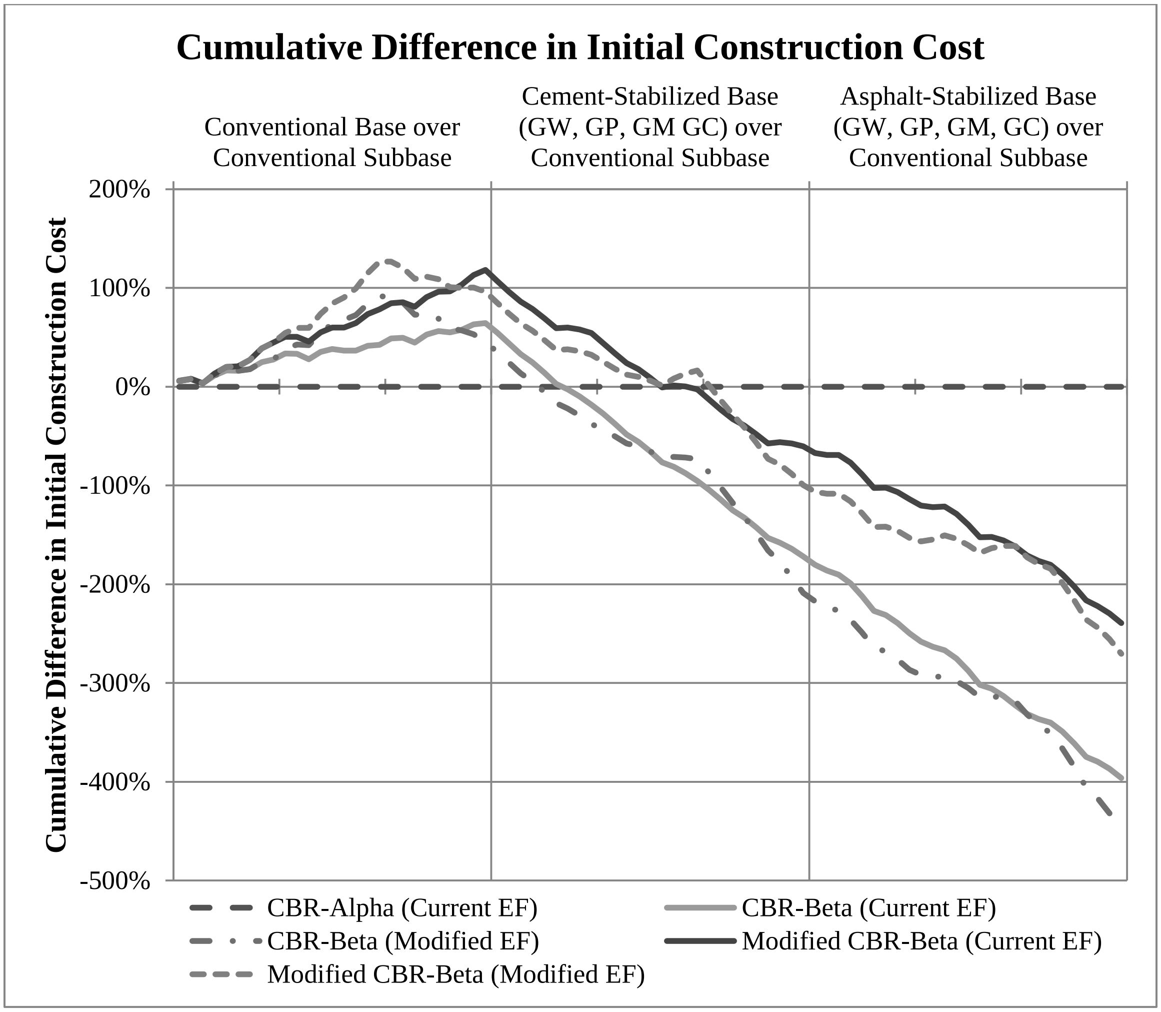


Figure 97. Comparison of the Cumulative Differences in Initial Construction Cost for the Modified CBR-Beta Design Method using the Two-Layer Concentration Factor Model – Aggregated Results (n = 81)

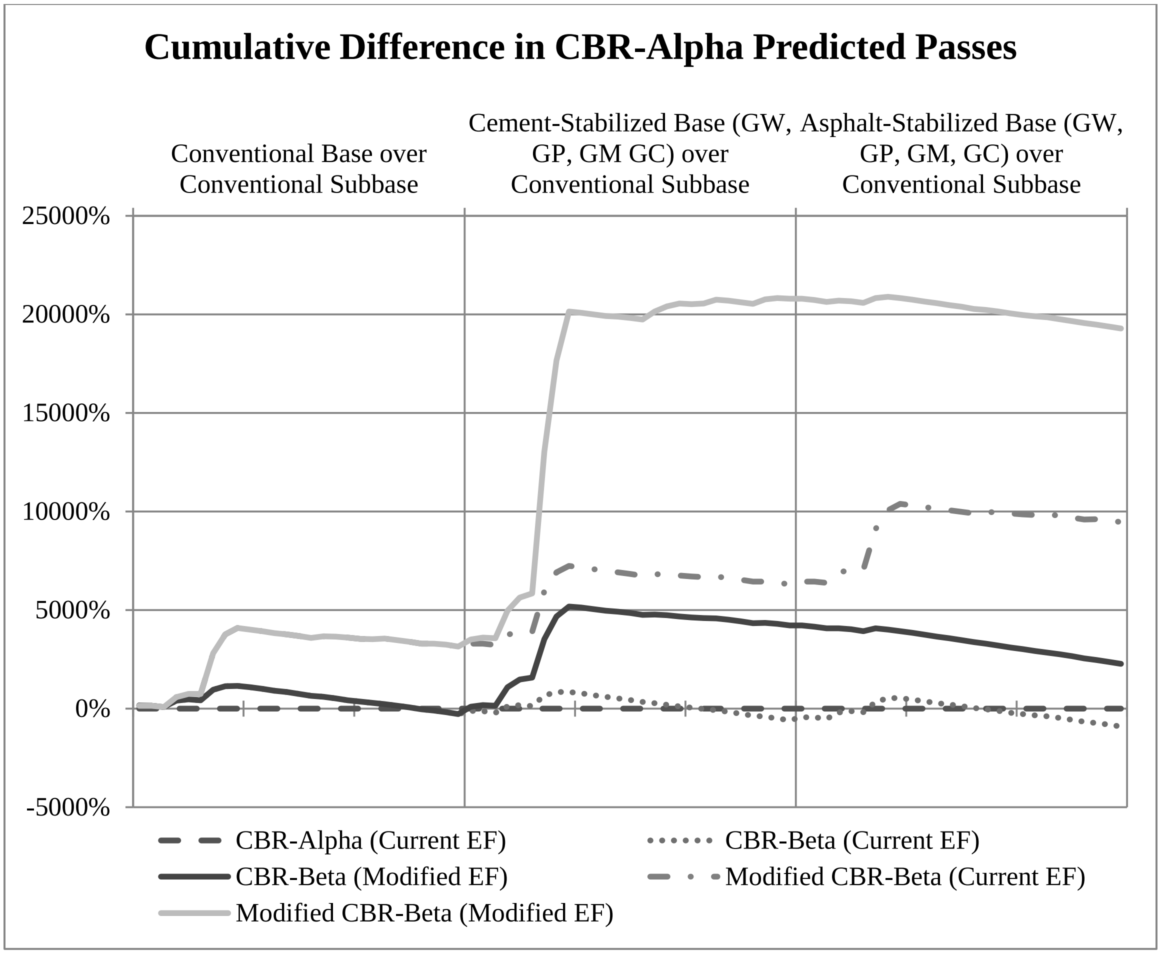


Figure 98. Comparison of the Cumulative Differences in Predicted CBR-Alpha Coverages Based on the Predicted Thickness Calculated by the Modified CBR-Beta Design Method using the Two-Layer Concentration Factor Model – Aggregated Results (n = 81)

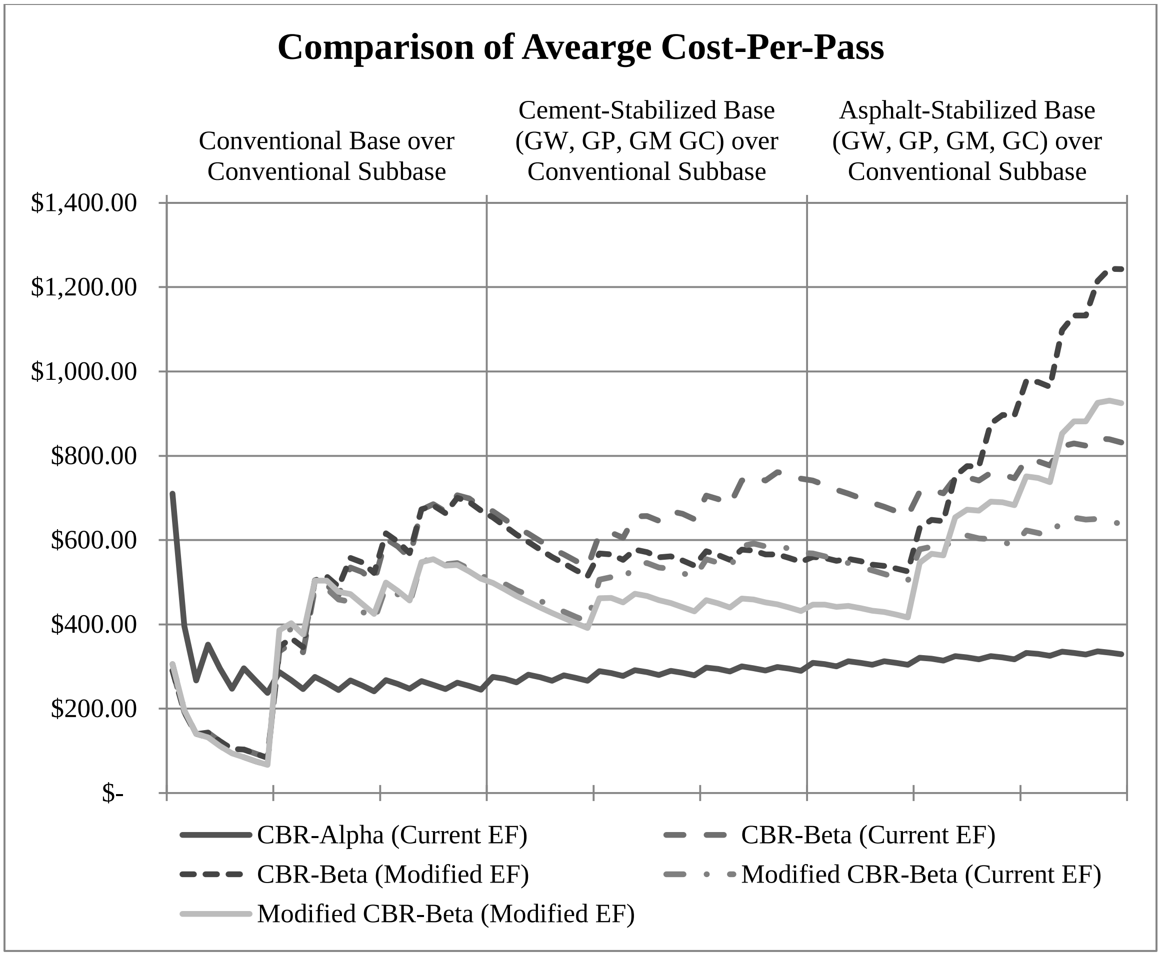


Figure 99. Comparison of the Cumulative Averages of the Cost per CBR-Alpha Predicted Pass Based on the Predicted Thickness Calculated by the Modified CBR-Beta Design Method using the Two-Layer Concentration Factor Model – Aggregated Results (n = 81)

Appendix H. Summary of Airfield Test Sections Used During this Study

All of the airfield test section incorporated into this study came from three sources: the USACE, the FAA, and Airbus. As with any meta-analysis using data from multiple testing agencies, due diligence was necessary to ensure that only data that had similar testing methodologies and failure criteria were utilized. Pavements were considered failed when rutting exceeded one inch; the coverages to failure were recorded using this distress condition. When not explicitly stated in the experimentation report, the failure coverages were interpolated from the reports using the cross-sectional profiles and deformation curves as applicable; test sections that did not fail under trafficking were included in the analysis, but received additional scrutiny to ensure the results did not result in extreme outliers.

Stabilized Test Sections

The commentary in the subsequent sections are reprinted in its entirety from the commentary on the same subject contained in Chapter III. This was done to provide further context to the reader in lieu of simply printing the tables herein this appendix.

Asphalt-stabilized

As shown in Table 24, the test section data collected for the asphalt stabilization methods aligned into two broad categories: (1) asphalt-stabilized GW, GP, GM, GC base course, and (2) asphalt-stabilized SW, SP, SM, SC subbase course. These two categories correspond to two of the five equivalency factor categories listed in Table 6 under asphalt-stabilized; no studies were found that support the other three factors. The test sections incorporated in this study involved various combinations of wheel assemblies and loads corresponding to contact pressures ranging from 105 to 278 pounds per square inch. These pressures were placed over flexible pavement structures with thicknesses above the subgrade ranging from approximately 12.6 to 39.5 inches; the subgrade CBRs ranged from 2.5 to 15.

All of the sources provided relevant data to this study; however, the Airbus source required some engineering judgment to extract acceptable data for this study. In their report, Martin et al. (2001) documented several instances where the test sections experienced immediate settlement under the pavement at the introduction of loading. With a lack of information as to the cause of this condition, eight test samples with unusually high settlement were omitted from this analysis as to avoid adversely affecting the overall results. This omission is deemed acceptable as the settling reached upwards of 1.2 inches, and it is unclear as to whether the settlement was construction or materials related. Additionally, the failure coverages from the report for the extracted test sections were interpolated from the rut versus passes graphs provided by the authors.

As an additional commentary about the data set, the four stabilized subbase test sections also contained asphalt-stabilized base courses; these tests are identified by the forward-slashed texture on Table 24. As a result of the dual stabilized layers, these tests were not included in the base course analysis; however, they were included in the subbase analysis using an equivalency factor to account for the base course stabilization. For this case, the equivalency factor for the subbase was determined from incorporating the equivalency factor calculated in this research into the equivalent thickness formulation to account for the stabilized base course.

According to the test reports, a majority of the FAA and Airbus test sections for the asphalt-stabilized base course material contained a crushed aggregate/gravel subbase course (Airbus, 2010; Gopalakrishnan & Thompson, 2004). For the FAA test section (identified by the dotted texture in Table 24), the subbases contained P-209, which meets the USACE's gradation requirements for base course materials. This subbase material represents an improvement over the conventional subbase material, and the U.S. Air Force and Army account for this material in its current set of equivalency factors with a factor of 2.0 (as shown in Table 6). On the other hand, the test sections from the Airbus tests contained a crushed gravel that did not meet the USACE's gradation requirements for base course materials; therefore, no equivalency factor was necessary to account for this material.

Table 24. Full-Scale Asphalt-Stabilized Test Sections (Flexible Airfield)

		Reference	Testing Agency	Wheel Type	Load (k)	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
						Wearing	Base	Subbase			Predicted	Actual
Base Course	Asphalt Stabilized GW, GP, GM, GC	1	Airbus	Twin Tandem	146	2.4	7.9	15.8	8.0	6105	33.32	25.98
		1	Airbus	Twin Tandem	146	3.2	7.1	15.8	8.0	6105	33.32	25.99
		1	Airbus	Twin Tandem	146	4.7	5.5	15.8	8.0	6395	33.42	25.98
		1	Airbus	Twin Tandem	146	3.2	7.1	15.8	8.0	6105	33.32	25.99
		1	Airbus	Twin Tandem	146	3.2	7.1	15.8	8.0	6395	33.42	25.99
		1	Airbus	Twin Tandem	146	3.2	7.1	15.8	8.0	5814	33.21	25.99
		2	Airbus	Twin Tandem	252	3.1	9.4	0.0	15.0	2148	20.35	12.60
		2	Airbus	Triple Tandem	310	3.1	9.4	0.0	15.0	3574+	18.83	12.60
		2	Airbus	Triple Tandem	378	3.1	9.4	0.0	15.0	901	19.29	12.60
		2	Airbus	Twin Tandem	207	3.1	9.4	0.0	15.0	2487	18.66	12.60
		2	Airbus	Twin Tandem	252	3.1	9.4	7.9	10.0	3464+	25.94	20.47
		2	Airbus	Triple Tandem	310	3.1	9.4	7.9	10.0	3574+	23.59	20.47
		2	Airbus	Triple Tandem	378	3.1	9.4	7.9	10.0	3464+	26.04	20.47
		2	Airbus	Twin Tandem	207	3.1	9.4	7.9	10.0	2646+	23.37	20.47
		3	USACE	Single Wheel	75	3.1	5.8	6.0	2.5	8	33.38	14.88
		3	USACE	Single Wheel	75	3.0	7.2	13.7	5.2	90	29.95	23.88
		3	USACE	Single Wheel	75	2.8	5.9	15.0	3.0	12	32.12	23.64
		3	USACE	12-Wheel Assembly	360	3.1	5.8	6.0	2.5	425	36.57	14.88
		3	USACE	12-Wheel Assembly	360	3.0	7.2	13.7	5.2	2198	24.16	23.88
		3	USACE	12-Wheel Assembly	360	2.8	5.9	15.0	3.0	734	33.32	23.64
		4	FAA	Triple Tandem	270	5.0	4.9	29.6	4.0	16359	44.19	39.51
		4	FAA	Triple Tandem	270	5.0	4.9	29.6	4.0	15358	43.92	39.51
		4	FAA	Triple Tandem	270	5.0	4.9	8.5	8.0	4674	24.83	18.38
		4	FAA	Triple Tandem	270	5.0	4.9	8.5	8.0	8891	25.72	18.38
		4	FAA	Twin Tandem	180	5.0	4.9	29.6	4.0	13841	42.75	39.51
		4	FAA	Twin Tandem	180	5.0	4.9	29.6	4.0	11372	42.18	39.51
		4	FAA	Twin Tandem	180	5.0	4.9	8.5	8.0	10515	25.91	18.38
		4	FAA	Twin Tandem	180	5.0	4.9	8.5	8.0	7996	25.55	18.38
Subbase Course	Asphalt Stabilized SW, SP, SM, SC	3	USACE	Single Wheel	75	3.1	5.8	6.0	2.5	8	33.38	14.88
		3	USACE	Single Wheel	75	3.0	7.2	13.7	5.2	90	29.95	23.88
		3	USACE	12-Wheel Assembly	360	3.1	5.8	6.0	2.5	425	36.57	14.88
		3	USACE	12-Wheel Assembly	360	3.0	7.2	13.7	5.2	2198	24.16	23.88

References
1. High Pressure Tire Test (Airbus, 2010)
2. A380 Pavement Experimental Programme (Martin, et al., 2001)
3. Study of Behavior of Bituminous-Stabilized Layers (Burns, Ledbetter, & Grau, 1973)
4. Rutting Study of NAPTF Flexible Pavement Test Sections (Gopalakrishnan & Thompson, 2004)

Cement-Stabilized

All of the data for the cement-stabilized factors came from the USACE testing data of which the vast majority came from a single report/experiment as shown in Table 25. This test included both channelized and distributed load patterns for the single wheel load cart. Both tests were included in this study; however, the pass-to-coverage ratios were adjusted to account for this variation. Additionally, these tests, for the most part, did not have a wearing course or a subbase course as is typically found in airfield pavements. As suggested in an internal USACE report, the test sections without a wearing course were adjusted to create an imaginary wearing course for the purpose of analysis by subtracting the minimum wearing course thickness from the predicted thickness prior to determining the base course equivalency factor (Barker, Gonzalez, Harrison, & Bianchini, 2012). Overall, the data set contained a variety of tests that included high-pressure single wheel and lower-pressure 12-wheel assemblies.

Other Stabilization Methods

Of the additional stabilized test sections not already mentioned, only two align with categories in Table 6: lime-stabilized ML, MH, CL, CH (five samples) and Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH (two samples). These test sections are shown in Table 26. The studies involving these two materials were completed by the USACE. However, definitive conclusions would be unreasonable for the lime, cement, fly ash stabilized equivalency factor, as only two test sections are available for analysis.

As previously mentioned, crushed aggregate that meets the gradation requirements for base course materials are accounted for with an equivalency factor of 2.0 when used in the subbase under the U.S. Air Force and Army's design methodology.

However, the FAA accounts for this improved material using an equivalency factor of approximately 1.4 (Federal Aviation Administration, 1995). Crushed aggregate is stronger than conventional subbase materials; therefore, it is logical to assume that an equivalency factor is necessary. As a result, this study analyzed the data to validate the current factor; however, the analysis was difficult since all of the crushed aggregate test sections contained asphalt-stabilized base courses.

Table 25. Full-Scale Cement-Stabilized Test Sections (Flexible Airfield)

		Reference	Testing Agency	Wheel Type	Load (k)	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
						Wearing	Base	Subbase			Predicted	Actual
Base Course	Cement Stabilized GW, GP, SW, SP	1	USACE	Single Wheel	27	0.0	9.1	0.0	4.67	48	17.93	9.10
		1	USACE	Single Wheel	27	0.0	9.1	0.0	4.67	7	14.75	9.10
		1	USACE	Single Wheel	27	0.0	12.7	0.0	6.43	668	18.60	12.70
		1	USACE	Single Wheel	27	0.0	12.7	0.0	6.43	37	15.13	12.70
		1	USACE	Single Wheel	27	0.0	16.6	0.0	5.30	1000+	20.72	16.60
		1	USACE	Single Wheel	27	0.0	7.9	0.0	5.03	4	13.41	7.90
		1	USACE	Single Wheel	27	0.0	11.7	0.0	4.57	129	19.51	11.70
		1	USACE	Single Wheel	27	0.0	11.7	0.0	4.57	21	16.83	11.70
		1	USACE	Single Wheel	27	0.0	16.3	0.0	5.10	1000	21.07	16.30
		1	USACE	Single Wheel	27	0.0	8.4	0.0	5.43	18	15.31	8.40
		1	USACE	Single Wheel	27	0.0	8.4	0.0	5.43	74	17.34	8.40
		1	USACE	Single Wheel	27	0.0	11.7	0.0	5.47	120	17.91	11.70
		1	USACE	Single Wheel	27	0.0	11.7	0.0	5.47	242	18.79	11.70
		1	USACE	Single Wheel	27	1.4	11.2	0.0	5.60	636	19.72	12.60
		1	USACE	Single Wheel	27	1.0	11.2	0.0	5.60	67	16.97	12.20
		2	USACE	12-Wheel Assembly	360	2.8	21.8	0.0	3.70	10406+	35.08	24.60
		2	USACE	Single Wheel	75	2.8	21.8	0.0	3.70	200	36.75	24.60
		2	USACE	Twin Tandem	200	2.8	21.8	0.0	3.70	1810	42.89	24.60
		2	USACE	Single Wheel	75	2.8	21.8	0.0	3.70	120	35.51	24.60
		3	USACE	Twin Tandem	200	3.0	25.0	0.0	3.80	7820	47.43	28.00
		3	USACE	Twin Tandem	240	3.0	25.0	0.0	3.20	620	52.96	28.00
Subbase Course	Cement Stabilized ML, MH, CL, CH	2	USACE	12-Wheel Assembly	360	2.5	4.7	15.7	2.70	1200	37.96	22.96
		2	USACE	Twin Tandem	160	2.5	4.7	15.7	2.70	1000	42.60	22.96
		2	USACE	Single Wheel	50	2.5	4.7	15.7	2.70	120	32.75	22.96
	Cement Stabilized SC, SM	1	USACE	Single Wheel	27	1.0	3.9	11.0	5.43	1000+	20.50	15.90
		1	USACE	Single Wheel	27	1.0	3.9	11.0	5.43	217+	18.72	15.90

References
1. Performance Data for F-4 Load Cart Operations on Alternate Launch and Recovery Surfaces (Styron III, 1984)
2. Evaluation of Structural Layers in Flexible Pavements (Grau, 1973)
3. Comparative Performance of Structural Layers in Pavements, Vol II (Sale, Hutchinson, Ulery, Ladd, & Barker, 1977)

Table 26. Full-Scale Miscellaneous Stabilized Test Sections (Flexible Airfield)

		Reference	Testing Agency	Wheel Type	Load (k)	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
						Wearing	Base	Subbase			Predicted	Actual
Subbase Course	Lime Stabilized ML, MH, CL, CH	1	USACE	Single Wheel	27	1.4	3.6	13.1	4.60	419	20.99	18.10
		1	USACE	Single Wheel	27	1.0	3.6	13.1	4.60	217	20.15	17.70
		2	USACE	12-Wheel Assembly	360	2.3	4.8	15.4	4.20	198	22.90	22.44
		2	USACE	Twin Tandem	160	2.3	4.8	15.4	4.20	140	25.61	22.44
		2	USACE	Single Wheel	50	2.3	4.8	15.4	4.20	40	24.61	22.44
	Lime, Cement, Fly Ash Stabilized ML, MH, CL, CH	3	USACE	Twin Tandem	200	3.0	6.0	24.0	5.60	3660	32.38	33.00
		3	USACE	Twin Tandem	240	3.0	6.0	24.0	4.40	600	39.92	33.00
	Crushed Aggregate (P-209)	4	FAA	Triple Tandem	270	5.0	4.9	29.6	4.00	16359	44.19	39.51
		4	FAA	Triple Tandem	270	5.0	4.9	29.6	4.00	15358	43.92	39.51
		4	FAA	Triple Tandem	270	5.0	4.9	8.5	8.00	4674	24.83	18.38
		4	FAA	Triple Tandem	270	5.0	4.9	8.5	8.00	8891	25.72	18.38
		4	FAA	Twin Tandem	180	5.0	4.9	29.6	4.00	13841	42.75	39.51
		4	FAA	Twin Tandem	180	5.0	4.9	29.6	4.00	11372	42.18	39.51
		4	FAA	Twin Tandem	180	5.0	4.9	8.5	8.00	10515	25.91	18.38
		4	FAA	Twin Tandem	180	5.0	4.9	8.5	8.00	7996	25.55	18.38

References

1. Performance Data for F-4 Load Cart Operations on Alternate Launch and Recovery Surfaces (Styron III, 1984)
2. Evaluation of Structural Layers in Flexible Pavements (Grau, 1973)
3. Comparative Performance of Structural Layers in Pavements, Vol II (Sale, Hutchinson, Ulery, Ladd, & Barker, 1977)
4. Rutting Study of NAPTF Flexible Pavement Test Sections (Gopalakrishnan & Thompson, 2004)

Non-Stabilized Test Sections

This study incorporated 88 non-stabilized test sections into the overall study (Table 27 and Table 28). These test sections came from the Federal Aviation Administration (FAA) or the U.S. Army Corps of Engineers (USACE). Of special note are the test sections highlighted by the forward slashed texture on Table 28, which contained a stabilized base course that the current USACE criteria does not provide an equivalency factor. As such, these test sections were analyzed as if it were a non-stabilized section.

Table 27. Non-Stabilized Full-Scale Airfield Test Sections

Reference	Testing Agency	Wheel Type	Load (k)	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
				Wearing	Base	Subbase			Predicted	Actual
1	USACE	12-Wheel Assembly	360	2.8	20.9	0.0	3.9	5000	31.68	23.64
1	USACE	Twin Tandem	200	2.8	20.9	0.0	3.9	890	38.58	23.64
1	USACE	Single Wheel	75	2.8	20.9	0.0	3.9	50	32.51	23.64
2	USACE	12-Wheel Assembly	360	3.4	3.5	10.7	3.0	8	19.54	17.56
2	USACE	12-Wheel Assembly	360	4.7	7.2	15.1	4.5	104	20.57	26.98
2	USACE	12-Wheel Assembly	360	4.0	8.1	24.0	3.4	1500	32.09	36.06
2	USACE	12-Wheel Assembly	360	4.4	7.8	23.2	3.5	1500	31.28	35.44
2	USACE	12-Wheel Assembly	360	3.5	7.7	31.7	4.0	3850	30.24	42.88
2	USACE	Single Wheel	50	3.4	3.5	10.7	3.0	6	23.72	17.56
2	USACE	Single Wheel	50	4.7	7.2	15.1	4.5	200	26.97	26.98
2	USACE	Single Wheel	30	3.4	3.5	10.7	3.0	120	23.49	17.56
2	USACE	Single Wheel	30	4.7	7.2	15.1	4.5	450	21.20	26.98
2	USACE	Twin Tandem	240	4.0	8.1	24.0	3.4	40	35.63	36.06
2	USACE	Twin Tandem	240	4.4	7.8	23.2	3.5	40	34.79	35.44
2	USACE	Twin Tandem	240	3.5	7.7	31.7	4.0	280	39.84	42.88
3	USACE	Single Wheel	38	2.5	8.0	16.0	9.8	97	13.88	26.50
3	USACE	Single Wheel	38	3.5	7.0	17.0	9.9	259	15.14	27.50
3	USACE	Single Wheel	38	4.0	6.5	15.0	9.3	118	14.65	25.50
3	USACE	Single Wheel	30	4.0	6.5	13.5	9.1	10283	19.04	24.00
3	USACE	Single Wheel	38	5.0	6.0	14.5	9.4	530	16.55	25.50
3	USACE	Single Wheel	30	5.0	6.0	12.0	9.5	7794	18.45	23.00
4	USACE	Single Wheel	30	2.0	10.0	0.0	7.1	4545	17.91	12.00
4	USACE	Single Wheel	30	2.0	6.0	0.0	7.1	305	14.57	8.00
4	USACE	Single Wheel	30	2.0	18.0	0.0	2.5	514	27.35	20.00
4	USACE	Single Wheel	30	2.0	12.0	0.0	2.5	41	22.46	14.00
4	USACE	Single Wheel	30	2.0	14.0	0.0	2.5	48	22.81	16.00
5	USACE	Six Wheel	214	0.0	24.0	0.0	3.1	12640	43.37	24.00
5	USACE	Six Wheel	214	0.0	20.0	0.0	3.1	12640	43.37	20.00
5	USACE	Six Wheel	214	0.0	16.0	0.0	3.1	3648	39.68	16.00
5	USACE	Six Wheel	214	0.0	15.0	0.0	6.9	12640	24.56	15.00
5	USACE	Six Wheel	214	0.0	12.0	0.0	6.9	2976	22.52	12.00
5	USACE	Six Wheel	214	0.0	9.0	0.0	6.9	264	18.81	9.00
5	USACE	Six Wheel	214	0.0	12.0	0.0	6.9	3	10.46	12.00
6	USACE	Single Wheel	10	0.0	5.0	0.0	7.0	8	6.24	5.00

References
1. Evaluation of Structural Layers in Flexible Pavement (Grau, 1973)
2. Multiple-Wheel Heavy Gear Load Pavement Tests (Hammitt II, Hutchinson, & Rice, 1971)
3. Evaluation of Minimum Asphalt Concrete Thickness Criteria (Bell & Mason, 2008)
4. Geogrid Reinforced Base Courses for Flexible Pavements for Light Aircraft (Webster, 1993)
5. Determination of Semi-Prepared Airfield Pavement Structural Requirements for Supporting C-17 Aircraft Gear (Grogran, 1998)
6. A Limited Study of Effects of Mixed Traffic on Flexible Pavements (Brown, 1962)

Table 28. Non-Stabilized Full-Scale Airfield Test Sections (Continued)

Reference	Testing Agency	Wheel Type	Load (k)	Course Thickness (in)			Subgrade	Failure	Total Thickness (in)	
				Wearing	Base	Subbase	CBR	Coverages	Predicted	Actual
7	USACE	Twin Tandem	120	1.5	14.5	0.0	15.0	1500	13.37	16.00
7	USACE	Twin Tandem	120	1.5	14.5	0.0	5.0	90	19.46	16.00
7	USACE	Twin Tandem	120	1.5	14.5	0.0	12.0	310	13.57	16.00
7	USACE	Single Wheel	30	1.5	10.0	0.0	14.0	216	12.03	11.50
7	USACE	Single Wheel	30	1.5	10.5	0.0	7.0	178	16.99	12.00
7	USACE	Single Wheel	30	1.5	10.0	0.0	6.0	203	18.46	11.50
8	USACE	Twin Tandem	150	3.0	11.0	0.0	18.0	2000	12.74	14.00
8	USACE	Twin Tandem	150	3.0	17.0	0.0	15.0	2000	14.38	20.00
8	USACE	Twin Tandem	150	3.0	23.0	0.0	21.0	2000	11.38	26.00
8	USACE	Twin Tandem	200	3.0	11.0	0.0	22.0	1410	13.43	14.00
8	USACE	Twin Tandem	200	3.0	17.0	0.0	30.0	2800	11.24	20.00
8	USACE	Twin Tandem	200	3.0	23.0	0.0	25.0	2800	12.93	26.00
8	USACE	Dual	70	3.0	7.0	0.0	19.0	2000	10.33	10.00
8	USACE	Dual	70	3.0	12.0	0.0	16.0	2000	12.02	15.00
8	USACE	Dual	70	3.0	17.0	0.0	18.0	2000	10.87	20.00
8	USACE	Dual	100	3.0	7.0	0.0	35.0	978	9.06	10.00
8	USACE	Dual	100	3.0	12.0	0.0	25.0	1400	12.46	15.00
8	USACE	Dual	100	3.0	17.0	0.0	20.0	2650	15.11	20.00
9	USACE	Single Wheel	75	2.6	12.0	0.0	2.9	6	30.30	14.64
9	USACE	12-Wheel Assembly	360	2.6	12.0	0.0	2.9	98	27.45	14.64
10	USACE	Twin Tandem	200	3.0	25.0	0.0	5.4	3660	33.40	28.00
10	USACE	Twin Tandem	200	3.0	25.0	0.0	4.9	1380	33.28	28.00
10	USACE	Twin Tandem	240	3.0	25.0	0.0	4.0	340	40.68	28.00
10	USACE	Twin Tandem	240	3.0	25.0	0.0	5.2	120	29.70	28.00
11	FAA	Triple Tandem	270	5.1	7.8	36.4	4.0	15447	43.94	49.26
11	FAA	Triple Tandem	270	5.1	7.8	36.4	4.0	15070	43.84	49.26
11	FAA	Triple Tandem	270	5.1	7.9	12.1	8.0	2389	23.88	25.14
11	FAA	Triple Tandem	270	5.1	7.9	12.1	8.0	853	22.37	25.14
11	FAA	Twin Tandem	180	5.1	7.8	36.4	4.0	11333	42.17	49.26
11	FAA	Twin Tandem	180	5.1	7.8	36.4	4.0	12045	42.35	49.26
11	FAA	Twin Tandem	180	5.1	7.9	12.1	8.0	631	21.93	25.14
11	FAA	Twin Tandem	180	5.1	7.9	12.1	8.0	766	22.22	25.14
12	FAA	Triple Tandem	270	5.0	8.0	16.0	3.7	64	28.32	29.00
12	FAA	Triple Tandem	270	5.0	8.0	24.0	4.4	1132	32.93	37.00
12	FAA	Triple Tandem	270	5.0	8.0	34.0	4.4	14295	39.63	47.00
12	FAA	Twin Tandem	180	5.0	8.0	16.0	4.3	70	25.94	29.00
12	FAA	Twin Tandem	180	5.0	8.0	24.0	4.3	1572	33.83	37.00
12	FAA	Twin Tandem	180	5.0	8.0	34.0	4.3	21170	41.37	47.00
13	FAA	Twin Tandem	220	2.5	8.0	16.0	3.3	60	35.64	26.50
13	FAA	Twin Tandem	220	2.5	8.0	24.0	3.4	908	48.17	34.50
13	FAA	Twin Tandem	220	2.5	8.0	16.0	3.3	29	31.96	26.50
13	FAA	Twin Tandem	220	2.5	8.0	24.0	3.1	1118	52.99	34.50
14	USACE	Single Wheel	54	3.0	6.0	7.0	14.2	144	15.06	16.00
14	USACE	Single Wheel	48	3.0	6.0	14.0	9.8	480	18.95	23.00
14	USACE	Single Wheel	48	3.0	6.0	16.0	3.9	160	27.60	25.00
14	USACE	Single Wheel	45	3.0	6.0	23.0	4.0	1600	30.26	32.00
14	USACE	Dual	90	3.0	6.0	7.0	14.2	1056	15.69	16.00
14	USACE	Dual	90	3.0	6.0	14.0	9.8	5664	21.66	23.00
14	USACE	Dual	90	3.0	6.0	16.0	3.9	320	30.18	25.00
14	USACE	Dual	90	3.0	6.0	23.0	4.0	1200	33.20	32.00
14	USACE	Single Wheel	36	3.0	6.0	7.0	14.2	73	12.86	16.00
14	USACE	Single Wheel	35	3.0	6.0	14.0	9.8	182	16.26	23.00
14	USACE	Single Wheel	35	3.0	6.0	16.0	3.9	18	20.48	25.00
14	USACE	Single Wheel	40	3.0	6.0	23.0	4.0	240	26.46	32.00

References

7. Investigation of Effects of Traffic with High-Pressure Tires on Asphalt Pavements (U.S. Army Waterways Experiment Station, 1950)
8. Design of Flexible Airfield Pavements for Multiple-Wheel Landing Gear Assemblies: Test Section with Lean Clay Subgrade (U.S. Army Waterways Experiment Station, 1952)
9. Study of Behavior of Bituminous-Stabilized Layers (Burns, Ledbetter, & Grau, 1973)
10. Comparative Performance of Structural Layers in Pavements, Vol II (Sale, Hutchinson, Ulery, Ladd, & Barker, 1977)
11. Rutting Study of NAPTF Flexible Pavement Test Sections (Gopalakrishnan & Thompson, 2004)
12. Traffic Testing Results from the FAA's National Airport Pavement Test Facility (Hayhoe, 2004)
13. Performance of Flexible Pavements over Two Subgrades with Similar CBR but Different Soil Types (Silty Clay and Clay) at the FAA's National Airport Pavement Test Facility (Garg & Hayhoe, 2008)
14. Reformulation of the CBR Procedure, Report II (Gonzalez, Barker, & Bianchini, 2013)

Appendix I. Summary of Highway Test Sections Used During this Study

The research contained in Chapter VI analyzed 123 non-stabilized, highway test sections. These sections were incorporated from data provided by the Minnesota Department of Transportation (MnROAD Test Facility) and the Highway Research Board (AASHO Road Tests) (Minnesota Department of Transportation, 2012; Highway Research Board, 1962b). The characteristics of the each test section are summarized in Table 29, Table 30, and Table 31. As mentioned in Chapter VI, the research team converted all of the vehicles and subsequent trafficking information into 18-kip equivalent single axle wheel loads (ESALs). This step was necessary as the trafficking data provided in the MnROAD tests was given in ESALs. Additionally, the subgrade CBR for the MnROAD tests were estimated using an empirical relationship because the subgrade strength was reported in terms of resilient modulus.

Table 29. Non-Stabilized Full-Scale Highway Test Sections

Reference	Testing Agency	Wheel Type	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
			Wearing	Base	Subbase			Predicted	Actual
1	MnROAD	18K ESAL	3.1	4.0	0.0	15.4	34380	10.42	7.10
1	MnROAD	18K ESAL	3.1	4.0	0.0	15.4	35037	10.46	7.10
1	MnROAD	18K ESAL	3.1	4.0	0.0	15.4	35682	10.49	7.10
1	MnROAD	18K ESAL	3.1	4.0	0.0	15.4	37309	10.58	7.10
1	MnROAD	18K ESAL	3.1	4.0	0.0	15.4	42476	10.83	7.10
1	MnROAD	18K ESAL	3.1	4.0	0.0	15.4	44087	10.91	7.10
1	MnROAD	18K ESAL	4.0	12.0	0.0	9.6	2155	9.63	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	9.6	4439	11.04	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	9.6	4524	11.08	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	9.6	5382	11.44	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	9.6	7345	12.09	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	9.6	9361	12.61	16.00
1	MnROAD	18K ESAL	4.0	6.0	0.0	10.0	2361	8.96	10.00
1	MnROAD	18K ESAL	4.0	6.0	0.0	10.0	2438	9.02	10.00
1	MnROAD	18K ESAL	4.0	6.0	0.0	10.0	2910	9.34	10.00
1	MnROAD	18K ESAL	4.0	6.0	0.0	10.0	4521	10.16	10.00
1	MnROAD	18K ESAL	4.0	6.0	0.0	10.0	8304	11.36	10.00
1	MnROAD	18K ESAL	4.0	6.0	0.0	10.0	9136	11.55	10.00
1	MnROAD	18K ESAL	4.0	6.0	0.0	10.0	11842	12.09	10.00
1	MnROAD	18K ESAL	5.1	10.0	0.0	9.6	32313	15.44	15.10
1	MnROAD	18K ESAL	5.1	10.0	0.0	9.6	37204	15.82	15.10
1	MnROAD	18K ESAL	5.1	12.0	0.0	7.9	32313	17.83	17.10
1	MnROAD	18K ESAL	5.1	12.0	0.0	7.9	35037	18.06	17.10
1	MnROAD	18K ESAL	5.1	12.0	0.0	7.9	37204	18.23	17.10
1	MnROAD	18K ESAL	5.1	12.0	0.0	7.9	37309	18.24	17.10
1	MnROAD	18K ESAL	4.0	4.0	0.0	8.5	7421	13.49	8.00
1	MnROAD	18K ESAL	4.0	4.0	12.0	8.5	7735	7.69	20.00
1	MnROAD	18K ESAL	4.0	4.0	12.0	8.5	13957	8.57	20.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	7.3	13550	16.20	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	7.3	18125	16.99	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	7.3	18836	17.10	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	7.3	19269	17.16	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	7.3	21053	17.40	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	7.3	21130	17.41	16.00
1	MnROAD	18K ESAL	4.0	12.0	0.0	7.3	21492	17.46	16.00
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	13550	16.24	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	15183	16.55	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	15864	16.67	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	18232	17.05	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	18836	17.13	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	19269	17.20	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	21053	17.44	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	21492	17.50	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	7.3	24609	17.87	15.90

References

1. MnROAD Data (Minnesota Department of Transportation, 2012)

Table 30. Non-Stabilized Full-Scale Highway Test Sections (Continued)

Reference	Testing Agency	Wheel Type	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
			Wearing	Base	Subbase			Predicted	Actual
1	MnROAD	18K ESAL	3.9	12.0	0.0	10.4	13550	12.68	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	10.4	15183	12.93	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	10.4	15864	13.02	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	10.4	18232	13.34	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	10.4	18836	13.41	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	10.4	19269	13.46	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	10.4	21053	13.66	15.90
1	MnROAD	18K ESAL	3.9	12.0	0.0	10.4	21130	13.67	15.90
2	AASHO	18K ESAL	1.0	6.0	4.0	3.0	992	9.45	11.00
2	AASHO	18K ESAL	2.0	6.0	4.0	3.0	8315	12.50	12.00
2	AASHO	18K ESAL	2.0	6.0	8.0	3.0	46977	17.20	16.00
2	AASHO	18K ESAL	3.0	3.0	8.0	3.0	15798	13.44	14.00
2	AASHO	18K ESAL	3.0	6.0	4.0	3.0	41483	15.28	13.00
2	AASHO	18K ESAL	4.0	0.0	8.0	3.0	11022	11.45	12.00
2	AASHO	18K ESAL	4.0	3.0	8.0	3.0	48036	15.24	15.00
2	AASHO	18K ESAL	4.0	3.0	8.0	3.0	47497	15.21	15.00
2	AASHO	18K ESAL	4.0	6.0	4.0	3.0	72961	15.94	14.00
2	AASHO	18K ESAL	3.0	0.0	12.0	3.0	61846	16.69	15.00
2	AASHO	18K ESAL	3.0	0.0	12.0	3.0	61171	16.67	15.00
2	AASHO	18K ESAL	3.0	6.0	12.0	3.0	260271	21.46	21.00
2	AASHO	18K ESAL	4.0	3.0	8.0	3.0	32729	14.40	15.00
2	AASHO	18K ESAL	4.0	3.0	8.0	3.0	29612	14.18	15.00
2	AASHO	18K ESAL	4.0	6.0	4.0	3.0	63899	15.64	14.00
2	AASHO	18K ESAL	5.0	0.0	12.0	3.0	305291	19.29	17.00
2	AASHO	18K ESAL	5.0	0.0	8.0	3.0	16939	11.79	13.00
2	AASHO	18K ESAL	5.0	6.0	4.0	3.0	169387	17.31	15.00
2	AASHO	18K ESAL	5.0	6.0	4.0	3.0	21819	12.79	15.00
2	AASHO	18K ESAL	3.0	9.0	4.0	3.0	73837	17.33	16.00
2	AASHO	18K ESAL	3.0	9.0	4.0	3.0	68646	17.15	16.00
2	AASHO	18K ESAL	3.0	3.0	12.0	3.0	96911	18.36	18.00
2	AASHO	18K ESAL	3.0	3.0	12.0	3.0	128195	19.08	18.00
2	AASHO	18K ESAL	3.0	6.0	8.0	3.0	50763	16.61	17.00
2	AASHO	18K ESAL	5.0	3.0	8.0	3.0	129215	16.91	16.00
2	AASHO	18K ESAL	5.0	3.0	8.0	3.0	144736	17.18	16.00
2	AASHO	18K ESAL	5.0	6.0	4.0	3.0	126541	16.62	15.00
2	AASHO	18K ESAL	4.0	6.0	12.0	3.0	574083	22.92	22.00
2	AASHO	18K ESAL	4.0	6.0	12.0	3.0	366389	21.58	22.00
2	AASHO	18K ESAL	3.0	3.0	12.0	3.0	55378	16.97	18.00
2	AASHO	18K ESAL	4.0	9.0	4.0	3.0	125523	17.92	17.00
2	AASHO	18K ESAL	4.0	9.0	4.0	3.0	125714	17.92	17.00
2	AASHO	18K ESAL	3.0	9.0	12.0	3.0	694068	24.84	24.00
2	AASHO	18K ESAL	4.0	3.0	12.0	3.0	130138	19.33	19.00
2	AASHO	18K ESAL	4.0	3.0	12.0	3.0	114135	18.99	19.00
2	AASHO	18K ESAL	4.0	6.0	8.0	3.0	127369	18.14	18.00
2	AASHO	18K ESAL	4.0	6.0	8.0	3.0	353983	20.86	18.00
2	AASHO	18K ESAL	4.0	6.0	8.0	3.0	136598	18.32	18.00
2	AASHO	18K ESAL	5.0	6.0	8.0	3.0	689453	21.98	19.00

References
1. MnROAD Data (Minnesota Department of Transportation, 2012)
2. The AASHO Road Test, Report 5: Pavement Research (Highway Research Board, 1962b)

Table 31. Non-Stabilized Full-Scale Highway Test Sections (Continued, Part 2)

Reference	Testing Agency	Wheel Type	Course Thickness (in)			Subgrade CBR	Failure Coverages	Total Thickness (in)	
			Wearing	Base	Subbase			Predicted	Actual
2	AASHO	18K ESAL	3.0	9.0	8.0	3.0	96911	18.64	20.00
2	AASHO	18K ESAL	3.0	9.0	8.0	3.0	119924	19.20	20.00
2	AASHO	18K ESAL	5.0	9.0	4.0	3.0	637767	21.55	18.00
2	AASHO	18K ESAL	5.0	9.0	4.0	3.0	580598	21.28	18.00
2	AASHO	18K ESAL	5.0	9.0	4.0	3.0	514090	20.93	18.00
2	AASHO	18K ESAL	5.0	9.0	4.0	3.0	505336	20.88	18.00
2	AASHO	18K ESAL	3.0	6.0	12.0	3.0	373833	22.53	21.00
2	AASHO	18K ESAL	4.0	6.0	16.0	3.0	1134727	25.57	26.00
2	AASHO	18K ESAL	6.0	3.0	16.0	3.0	1925837	25.13	25.00
2	AASHO	18K ESAL	5.0	6.0	8.0	3.0	238978	19.04	19.00
2	AASHO	18K ESAL	6.0	9.0	8.0	3.0	1895660	24.80	23.00
2	AASHO	18K ESAL	6.0	3.0	8.0	3.0	521238	19.79	17.00
2	AASHO	18K ESAL	6.0	3.0	8.0	3.0	863997	21.19	17.00
2	AASHO	18K ESAL	4.0	3.0	12.0	3.0	1056193	24.35	19.00
2	AASHO	18K ESAL	4.0	3.0	12.0	3.0	818875	23.54	19.00
2	AASHO	18K ESAL	5.0	6.0	12.0	3.0	1116344	24.11	23.00
2	AASHO	18K ESAL	5.0	6.0	12.0	3.0	1988934	26.03	23.00
2	AASHO	18K ESAL	5.0	6.0	12.0	3.0	975965	23.68	23.00
2	AASHO	18K ESAL	5.0	9.0	8.0	3.0	1602121	25.14	22.00
2	AASHO	18K ESAL	5.0	9.0	8.0	3.0	1154781	24.07	22.00
2	AASHO	18K ESAL	5.0	3.0	16.0	3.0	1344245	24.85	24.00
2	AASHO	18K ESAL	5.0	3.0	16.0	3.0	1248366	24.60	24.00
2	AASHO	18K ESAL	4.0	3.0	16.0	3.0	502034	22.64	23.00
2	AASHO	18K ESAL	4.0	3.0	16.0	3.0	1134727	25.23	23.00
2	AASHO	18K ESAL	4.0	9.0	8.0	3.0	696879	23.37	21.00
2	AASHO	18K ESAL	4.0	6.0	12.0	3.0	244159	20.43	22.00
2	AASHO	18K ESAL	4.0	6.0	12.0	3.0	222266	20.17	22.00
2	AASHO	18K ESAL	6.0	6.0	8.0	3.0	397787	19.68	20.00
2	AASHO	18K ESAL	6.0	6.0	8.0	3.0	511379	20.36	20.00
2	AASHO	18K ESAL	4.0	3.0	16.0	3.0	309999	21.22	23.00
2	AASHO	18K ESAL	4.0	3.0	16.0	3.0	235635	20.45	23.00
2	AASHO	18K ESAL	6.0	3.0	12.0	3.0	419734	19.99	21.00

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2. The AASHO Road Test, Report 5: Pavement Research (Highway Research Board, 1962b)

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Vita

Captain Thomas M. Synovec graduated from the Honors School at Ridge View High School in Columbia, South Carolina. He then enrolled at the U.S. Air Force Academy and subsequently graduated in 2009 with a Bachelor of Science in Civil Engineering. Upon graduation, he received a commission in the Regular Air Force and was assigned to the 355th Civil Engineer Squadron at Davis-Mothan Air Force Base, Arizona. Captain Synovec worked in various roles during his tenure, to include as a Civil Engineer on the in-house Civil Design Team, as the Base Traffic and Sign Engineer, the Officer-in-Charge of Heavy Repair, and lastly as the Readiness and Emergency Management Flight Commander. During that assignment, Captain Synovec deployed in 2011 for seven months to Afghanistan to serve as a Design Engineer and Community Planner. Prior to leaving Davis-Monthan for his next assignment, Captain Synovec completed the last course to earn his Master of Civil Engineering degree with a focus in structural engineering from North Carolina State University.

Captain Synovec entered the Air Force Institute of Technology at Wright-Patterson Air Force Base, Ohio in August 2012. In March 2014, he graduated with a Master of Science in Engineering Management; he was selected for a follow-on assignment at the 823rd RED HORSE Squadron, Hurlburt Field, Florida. Captain Synovec is a registered Professional Engineer (Civil) in the state of Arizona, as well as a Leadership in Energy and Environmental Design (LEED) Accredited Professional.

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14. ABSTRACT
With over 1.5 billion square feet of airfield pavements in its portfolio, the U.S. Air Force has a vested financial interest in refining its design, maintenance, and inspection criteria to increase the efficiency of its investment in its infrastructure. As part of its strategic pavement assessment, the Air Force is currently moving to adopt the new design method (CBR-Beta) developed by the U.S. Army Corps of Engineers (USACE) on the basis that the new methodology more accurately represents the performance of flexible airfield pavements, particularly with newer, heavier aircraft. Supporting this adoption of the new method, this research primarily focused on evaluating the current set of equivalency factors in use by the Air Force and the USACE using a meta-analysis approach. Building on this initial success, the research shifted to analyzing the life-cycle costs of the various flexible pavement design methods relative to a common design standard to eliminate the problems associated with comparing the methods each with its own assumptions and processes. Looking to further refine the predictability of the CBR-Beta method, the research analyzed the formulation of O.K. Frohlich's concentration factor. Additionally, the research assessed the possibility of expanding the empirical airfield data set with highway testing data.

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